

Imperial College London
Department of Civil and Environmental Engineering

Testing and design of conventional and novel stainless steel hollow structural sections

Craig Buchanan

February 2017

Submitted in part fulfilment of the requirements for the degree of Doctor of Philosophy in Civil and Environmental Engineering of Imperial College London and the Diploma of Imperial College London

Declaration of originality

I confirm that this thesis is my own work and that any material from published or unpublished work from others is appropriately referenced.

Copyright declaration

The copyright of this thesis rests with the author and is made available under a Creative Commons Attribution Non-Commercial No Derivatives licence. Researchers are free to copy, distribute or transmit the thesis on the condition that they attribute it, that they do not use it for commercial purposes and that they do not alter, transform or build upon it. For any reuse or redistribution, researchers must make clear to others the licence terms of this work.

Abstract

The topic of this thesis is the testing and design of conventionally formed and additive manufactured stainless steel hollow structural sections. Although design codes currently exist for stainless steel hollow structural elements, the provisions are based on limited structural data and therefore require further evaluation for their suitability, and are not intended to apply to additive manufactured elements. Focussing on conventionally formed circular hollow sections (CHS), the existing design provisions have been carefully reappraised based on a dataset of tests and finite element results generated in this study and existing tests collected from the literature. In total, 37 concentrically loaded column tests, 26 beam-column tests and 10 stub column tests have been undertaken on austenitic, duplex and ferritic stainless steel CHS. The experimental data has been supplemented with over 2400 finite element case studies. The reappraisal highlighted that there is additional capacity to be sought at the cross-section level for pure compression, bending and combined loading, and at the member level for beam-columns, but the current CHS flexural buckling provisions were found to be unconservative for certain global slenderness values. Based on these observations, revised design rules have been proposed.

Additive manufactured sections, not currently covered by structural design standards, have also been investigated. An experimental programme consisting of 28 tensile coupon tests, 14 compressive coupon tests and 5 square hollow section (SHS) stub column tests has been undertaken. The initial results indicate general applicability of existing design standards to these new novel sections.

Acknowledgements

This study was carried out under the supervision of Professor Leroy Gardner in the Department of Civil and Environmental Engineering at Imperial College London (ICL). I am grateful for his continued advice and encouragement over the past three years and for his enthusiasm to explore new areas of structural engineering research. I am also thankful to Professor Esther Real for the opportunity to undertake experiments in Barcelona, Spain, and for her guidance and support whilst working at Universitat Politècnica de Catalunya (UPC).

This project was possible thanks to funding and support from the Engineering and Physical Sciences Research Council (EPSRC), Outokumpu Research Foundation, Ministerio de Economía y Competitividad (Spanish Ministry of Economics and Competitiveness) under the Project BIA 2012-36373, an Erasmus+ Traineeship grant and EOS Finland.

I would also like to thank my international collaborators for their input and assistance, including Dr May Su and Professor Ben Young at the University of Hong Kong, Ville-Pekka Matilainen and Professor Antti Salminen at Lappeenranta University of Technology (LUT) and Fraunhofer Institute for Laser Technology (ILT). The collaboration with LUT was arranged with the help from Dr Mikko Vänskä following one of many thought provoking Outokumpu student seminars organised by Staffan Hertzman.

The experimental programme was possible thanks to assistance in the laboratory from Gordon Herbert, Trevor Stickland, Les Clark, Alfredo Olivo, Ron Millward, Bob Hewitt,

Hayden Spektor and Joel Thomas at ICL, Tomás García, Jordi Lafuente and Jordi Cabrerizo at UPC and staff at the LUT laser laboratory and machine shop. Aïa Ferrer and Jonathan Walsh also provided help in the laboratory as part of their respective undergraduate and MSc projects. I would like to recognise the significant contribution of Dr Itsaso Arrayago in the preparatory work for the experimental programme at UPC, and Dr Rolando Chacón and Professor Enrique Mirambell for their interest and support during the tests at UPC.

I am grateful to the numerous colleagues I have been fortunate to share an office with over the years, including Dr Sheida Afshan, Yidu Bu, Andreas Fieber, Dr Andrew Foster, Dr Jonathan Gosaye Fida Kaba, Nicolaos Hadjipantelis, (soon-to-be) Dr Pinelopi Kyvelou, Dr Andrew Liew, Dr Elizabeth Liu, Dr Finian McCann, Xin Meng, Dr Wei Qiu, Dr Marios Theofanous, Nathan Vella, Fiona Walport, Dr Jie Wang, Xiang Yun and Dr Ou Zhao for discussions related to my research and the ‘occasional conversations’ unrelated to structural engineering. I am also thankful for the wider departmental community for their help and support over the past few years, in particular to Fionnuala Ni Dhonnabhain for her humour and assistance in navigating the bureaucratic waters of the department, and to Filip Babovic, Dr Sophie Damy, Dr Geoff Fowler, Dr Andrew Phillips, Dr Sunday Popo-Ola, Andy Pullen, Ramandeep Singh, Jean Paul Vella and Professor Ahmer Wadee for their support and interest in my research.

I am also appreciative of the motivation and support from Thomas Ravalde, Li Ma, Dr Mark Bruggemann and George Goldberg, my former classmates from the 2012 undergraduate course who pursued their own research interests in the department, and to my housemates, Abi Attrell, Marcus Bishop, Thomas Beales Ferguson, Adam Ladds, Otso Liikanen, Oliver Mattos, Dr Charlotte Ridler, Dr Jacob Spencer and Richard Surgenor.

Finally I am grateful for the continued encouragement and support from my parents and brother throughout my studies.

Contents

Abstract	1
Acknowledgements	5
List of figures	13
List of tables	20
Notation	24
List of publications	33
1 Introduction	35
1.1 General background	35
1.2 Research aims	42
1.3 Outline of the thesis	42
2 Literature review	44
2.1 Circular hollow sections	44
2.2 Stainless steel	50
2.2.1 The ‘discovery’ of stainless steel	50
2.2.2 Types of stainless steel	50
2.2.3 Corrosion resistance	52
2.2.4 Comparisons with carbon steel	53
2.2.5 Stainless steel in construction	56
2.2.6 Existing stainless steel hollow section experiments	62
2.2.7 Stainless steel material models	64

2.3	Existing design codes and methods	68
2.3.1	Stainless steel design standards	68
2.3.2	The continuous strength method (CSM)	69
2.4	Finite element modelling	73
2.4.1	Residual stresses	74
2.4.2	Mesh elements and size	74
2.4.3	Geometric imperfections	75
2.4.4	Solver	76
2.5	Additive manufacturing	77
2.5.1	Introduction	77
2.5.2	Metallic additive manufacturing	78
2.5.3	Additive manufacturing in construction	82
2.6	Concluding remarks	83
3	Cross-section behaviour of CHS	87
3.1	Introduction	87
3.1.1	Traditional CHS design methods	87
3.1.2	The continuous strength method (CSM)	88
3.1.3	Extension of the CSM to CHS design	89
3.2	Cross-section slenderness	89
3.3	CHS experimental database	90
3.4	Stub column tests	91
3.4.1	General overview	91
3.4.2	Specimen preparation	93
3.4.3	Tensile coupon testing	99
3.4.4	Geometric properties	106
3.4.5	Stub column test setup	107
3.4.6	Stub column test results	109
3.5	Yield slenderness limit	111
3.6	Normalised deformation capacity (strain ratio)	116

3.6.1	Axial compression	117
3.6.2	Four-point bending	117
3.7	Base curve	119
3.8	Material models	121
3.8.1	Hot-finished structural carbon steel sections	121
3.8.2	Cold-formed structural carbon steel sections	122
3.8.3	Stainless steel sections	123
3.8.4	Aluminium sections	124
3.8.5	Summary of material models	124
3.9	CSM cross-section resistance functions for CHS	125
3.9.1	Compressive resistance	125
3.9.2	Bending resistance	126
3.10	Comparison with test data and existing design methods	131
3.11	Conclusions	135
4	Flexural buckling of CHS compression members	136
4.1	Introduction	136
4.2	Flexural buckling tests	137
4.2.1	General overview	137
4.2.2	Material properties	138
4.2.3	Geometric properties	138
4.2.4	Stub column tests	139
4.2.5	Flexural buckling test setup	140
4.2.6	Global imperfection and applied eccentricity	143
4.2.7	Flexural buckling test results	147
4.3	Finite element modelling	149
4.3.1	Geometry	154
4.3.2	Material model	155
4.3.3	Boundary conditions	156
4.3.4	Mesh elements and size	157

4.3.5	Geometric imperfections	157
4.3.6	Solver	159
4.3.7	Validation	159
4.3.8	Parametric study	166
4.4	Existing design methods	169
4.4.1	EN 1993-1-4 (2015) design provisions	171
4.4.2	Rasmussen and Rondal (2000) design provisions	173
4.4.3	Young and Ellobody (2006) design provisions	175
4.4.4	Theofanous et al. (2009b) design provisions	176
4.4.5	New flexural buckling curve proposal	176
4.5	Discussion of existing and new design provisions	176
4.6	Reliability analyses	178
4.7	Conclusions	182
5	CHS beam-columns	185
5.1	Introduction	185
5.2	Combined axial loading and bending moment and beam-column tests .	185
5.2.1	General overview	185
5.2.2	Material properties	186
5.2.3	Geometric properties	186
5.2.4	Stub column tests	188
5.2.5	Combined axial loading and bending moment test setup	188
5.2.6	Member beam-column test setup	190
5.2.7	Applied eccentricity	191
5.2.8	Combined axial loading and bending moment test results	192
5.2.9	Member beam-column test results	194
5.3	Finite element modelling	197
5.3.1	Boundary conditions	200
5.3.2	Validation	201
5.3.3	Parametric study	206

5.4	Comparisons with existing design provisions	206
5.4.1	EN 1993-1-4 (2015) design provisions for the cross-section behaviour	208
5.4.2	The continuous strength method design provisions	211
5.4.3	EN 1993-1-4 (2015) design provisions for the member behaviour	213
5.4.4	Proposed design provisions for the member behaviour	214
5.4.5	Discussion of existing and new design provisions	219
5.5	Reliability analyses	222
5.6	Conclusions	226
6	Additive manufacturing	228
6.1	Introduction	228
6.2	Experimental investigation	229
6.2.1	Powder bed fusion manufacturing	230
6.2.2	Tensile coupon tests	235
6.2.3	Compressive coupon tests	251
6.2.4	Stub column geometric properties	256
6.2.5	Stub column testing	257
6.3	Comparison with existing design methods	265
6.3.1	Comparisons with EN 1993-1-4 (2015) design provisions	265
6.3.2	Comparisons with the continuous strength method design provisions	267
6.4	Conclusions	269
7	Conclusions and suggestions for future work	271
7.1	Conclusions	271
7.2	Suggestions for further work	275
	References	278
A	ABAQUS Python script	307

A.1	Overview	307
A.2	Elastic buckling analysis	307
A.3	Nonlinear analysis	316

List of Figures

1.1	The Chrysler Building, New York with stainless steel cladding.	37
1.2	The stainless steel Cloud Gate sculpture in Millennium Park, Chicago, Illinois.	38
1.3	The United States Air Force Memorial in Arlington, Virginia.	39
1.4	The Helix Bridge, Singapore with a stainless steel CHS truss.	40
1.5	Stainless steel CHS in the entrance lobby of the Cordis Hotel, Mong Kok, Hong Kong.	41
2.1	The Forth Bridge, Queensferry, Scotland with CHS main compression members.	45
2.2	The Petronas Towers, Kuala Lumpur with a two-hinged CHS arch supporting the Skybridge.	46
2.3	The London Eye with a CHS triangle lattice compression rim.	47
2.4	Stratford Market Depot, Newham, London with a CHS truss supported roof.	49
2.5	The Connaught Tunnel cofferdam, Newham, London with CHS props supporting the dock walls.	49
2.6	The dome of St Paul's Cathedral, London with stainless steel circumferential bands.	58
2.7	The Lloyd's Building, London with 30,000 m ² of stainless steel cladding and pipework.	59
2.8	Stainless steel cladding on One Canada Square, London.	60
2.9	The Stonecutters Bridge, Hong Kong with a composite stainless steel skin on the upper portion of the main towers.	61

2.10 The test firing of an AM nickel superalloy engine chamber (SpaceX, 2014).	79
2.11 A test structure being formed through concrete printing (BBC News, 2016).	84
2.12 A plastic canal house development model (Frearson, 2016).	84
2.13 An MX3D flexural buckling test specimen (Joosten, 2015).	85
2.14 The progression from a conventionally fabricated to an optimised, additive manufactured lighting node (Arup, 2015).	86
3.1 The structures laboratory at Imperial College London.	92
3.2 The structures laboratory at Universitat Politècnica de Catalunya.	92
3.3 The uncut lengths of the 106×3 CHS with ovalised cross-sections.	95
3.4 The uncut lengths of the 106×3 CHS showing longitudinal deformations.	96
3.5 A CHS specimen being cut using a bandsaw, with wooden inserts inside and outside the specimen.	97
3.6 A short CHS specimen being machined, with the aluminium insert visible.	98
3.7 A long CHS specimen being machined.	98
3.8 A CHS coupon being machined, with the wooden block visible.	99
3.9 The 80×1.5-1 CHS tensile coupon in the testing machine, with strain measuring dots visible and black electrical tape reducing reflections.	101
3.10 The CHS end radius maintained within the testing machine grip.	102
3.11 Left to right, i) a CHS coupon end without filling material after a test, ii) a CHS coupon with filling material after test with the initial radius maintained and iii) a CHS coupon with filling material prior to testing.	103
3.12 Typical tensile coupon stress–strain curves.	104
3.13 The CHS grid painting setup.	107
3.14 The ICL stub column test setup.	108
3.15 The UPC stub column test setup.	109
3.16 Stub column load–end shortening curves.	111
3.17 The deformed 106×3-400-F, 106×3-400-FR, 104×2-400-F, 104×2-400- FR, 88.9×2.6-250-F and 88.9×2.6-250-FR CHS stub columns, from left to right.	113

3.18 The deformed 80×1.5 -350-F, 80×1.5 -350-FR, 101.6×1.5 -400-F and 101.6×1.5 -400-FR CHS stub columns, from left to right.	113
3.19 Normalised ultimate axial resistance N_u/N_y varying with local slenderness $\bar{\lambda}_c$	115
3.20 Normalised ultimate flexural resistance M_u/M_{el} varying with local slenderness $\bar{\lambda}_c$	116
3.21 The CHS CSM non-slender base curve with the collected experimental data.	119
3.22 The CHS CSM slender base curve with the collected experimental data.	120
3.23 The elastic, linear strain hardening CSM material model.	122
3.24 Strain and stress profiles for half of a CHS (a quarter of the cross-section is shown).	126
3.25 Geometry for the derivation of W_w for CHS.	127
3.26 Comparison between the exact and approximate (design) CSM bending capacity predictions M_{csm}/M_{pl} with varying $\varepsilon_{csm}/\varepsilon_y$	130
3.27 The influence of E_{sh}/E on M_{csm}/M_{pl} with varying $\varepsilon_{csm}/\varepsilon_y$	131
3.28 CSM and Eurocode compression resistance prediction comparison.	133
3.29 CSM and Eurocode bending resistance prediction comparison.	134
4.1 The ICL Instron 2000kN-SPL pin-ended column test setup.	141
4.2 The UPC Instron 8805 pin-ended short column test setup.	141
4.3 The UPC Ibertest MDA-700 pin-ended long column test setup.	142
4.4 The laser distance measurement sensor at UPC.	143
4.5 A flexural buckling test being filmed for later review.	144
4.6 An adjustable screw ram with a dial gauge at ICL.	146
4.7 Oversized bolt holes in an end plate at UPC.	147
4.8 The deformed austenitic 106×3 CHS long columns.	150
4.9 The deformed austenitic 104×2 CHS long columns.	151
4.10 The deformed duplex 88.9×2.6 CHS long columns.	152
4.11 The deformed ferritic 80×1.5 CHS long columns.	153

4.12 The deformed ferritic 101.6×1.5 CHS long columns.	153
4.13 The variation of the normalised ultimate load with the mesh size for the 106×3-1150-P CHS specimen.	158
4.14 The variation of the normalised mid-height deflection with the mesh size for the 106×3-1150-P CHS specimen.	158
4.15 Experimental and FE load mid-height lateral deformation curves for the 88.9×2.6-950-P CHS specimen.	164
4.16 Experimental and FE load mid-height lateral deformation curves for the 106×3-1650-P CHS specimen.	165
4.17 Experimental and FE failure modes for the 104×2-750-P CHS specimen.	165
4.18 Experimental and FE failure modes for the 106×3-3080-P CHS specimen.	166
4.19 Load–deformation history for the 120×2.6-1550-P CHS model with a full and half cross-section modelled.	169
4.20 Load–deformation history for the 65×1.5-4400-P CHS model with a full and half cross-section modelled.	171
4.21 Comparison between the experimental and numerical results and single buckling curves for the austenitic CHS dataset.	173
4.22 Comparison between the experimental and numerical results and single buckling curves for the duplex CHS dataset.	174
4.23 Comparison between the experimental and numerical results and single buckling curves for the ferritic CHS dataset.	174
4.24 Comparisons of the existing design provisions with experimental and numerical data for austenitic class 1–3 cross-sections.	178
4.25 Comparisons of the existing design provisions with experimental and numerical data for duplex class 1–3 cross-sections.	179
4.26 Comparisons of the existing design provisions with experimental and numerical data for ferritic class 1–3 cross-sections.	179
5.1 The short beam-column test setup.	189
5.2 The long beam-column test setup.	190

5.3	Load–end rotation plot for the 80×1.5 CHS short beam-column specimens.	192
5.4	Load–end rotation plot for the 101.6×1.5 CHS short beam-column specimens.	194
5.5	The deformed short beam-columns, with load eccentricity increasing from left to right.	195
5.6	End plate bending about the last line of bolts for the 80×1.5 -450-P-100E CHS specimen.	195
5.7	The deformed short beam-column with welded stiffeners on the end plates.	196
5.8	Load–mid height lateral deflection plot for the 80×1.5 CHS long beam-column specimens.	197
5.9	Load–mid height lateral deflection plot for the 101.6×1.5 CHS long beam-column specimens.	199
5.10	The deformed 80×1.5 CHS long beam-columns, with load eccentricity increasing from bottom to top.	199
5.11	The deformed 101.6×1.5 CHS long beam-columns, with load eccentricity increasing from bottom to top, including the unsuccessful test specimens.	200
5.12	Comparison between the 80×1.5 CHS cross-section test results under combined loading with the interaction curves.	210
5.13	Comparison between the 101.6×1.5 CHS cross-section test results under combined loading with the interaction curves.	210
5.14	Comparison of the 80×1.5 CHS beam-column member test results with the interaction curves.	214
5.15	Comparison of the 101.6×1.5 CHS beam-column member test results with the interaction curves.	215
5.16	k_{csm} factors determined from the numerical dataset for the austenitic specimens with a load level of $n = 0.2$	216
5.17	Proposed and calculated k_{csm} interaction factors for the austenitic beam-column specimens.	217
5.18	Comparison of the interaction curves at the cross-section level.	218

5.19 Definition of $N_{u,pred}$ on the moment-axial interaction curve.	219
5.20 Definition of θ on the moment-axial interaction curve.	222
5.21 Normalised resistance predictions for the austenitic stainless steel CHS beam-column dataset with the EN 1993-1-4 (2015) approach.	223
5.22 Normalised resistance predictions for the austenitic stainless steel CHS beam-column dataset with the new proposed approach.	223
5.23 Normalised resistance predictions for the duplex stainless steel CHS beam- column dataset with the EN 1993-1-4 (2015) approach.	224
5.24 Normalised resistance predictions for the duplex stainless steel CHS beam- column dataset with the new proposed approach.	224
5.25 Normalised resistance predictions for the ferritic stainless steel CHS beam- column dataset with the EN 1993-1-4 (2015) approach.	225
5.26 Normalised resistance predictions for the ferritic stainless steel CHS beam- column dataset with the new proposed approach.	225
6.1 Overview of the PBF process from the computer model to the final product.	232
6.2 The 316L stub columns during building.	233
6.3 The PH1 tensile coupons after building, still attached to the build platform.	233
6.4 The 316L stub columns after building, still attached to the build platform.	234
6.5 A cross-section through a typical support structure for a compression coupon.	234
6.6 An X-ray image of the PH1-1, PH1-2 and PH1-3 tensile coupons.	237
6.7 The nominal dimensions of the PH1 and 316L tensile coupons.	237
6.8 Building orientation angles θ , ψ and ϕ for the tensile and compressive coupons.	238
6.9 A fractured 316L tensile coupon at the end of a test.	239
6.10 Measured stress–strain curves from the tensile PH1 coupons.	244
6.11 Measured stress–strain curves from the tensile 316L coupons.	244
6.12 Normalised Young’s moduli varying with building orientation for the tensile PH1 coupons.	245

6.13 Normalised 0.2% proof stress varying with building orientation for the tensile PH1 coupons.	245
6.14 Normalised ultimate stress varying with building orientation for the tensile PH1 coupons.	246
6.15 Normalised fracture strain varying with building orientation for the tensile PH1 coupons.	246
6.16 Normalised Young's moduli varying with building orientation for the tensile 316L coupons.	247
6.17 Normalised 0.2% proof stress varying with building orientation for the tensile 316L coupons.	248
6.18 Normalised ultimate stress varying with building orientation for the tensile 316L coupons.	248
6.19 Normalised fracture strain varying with building orientation for the tensile 316L coupons.	249
6.20 The compression coupon setup with the nominal coupon dimensions. .	251
6.21 The compression coupon test setup.	252
6.22 Measured stress–strain curves from the compressive 316L coupons. . .	253
6.23 The local imperfection measurement setup.	258
6.24 Longitudinal imperfection profiles for 50×50×1 SHS specimen.	259
6.25 Longitudinal imperfection profiles for 50×50×5 SHS specimen.	259
6.26 The stub column testing setup.	261
6.27 Unprocessed rough surface finish due to the novel manufacturing process.	262
6.28 The digital image correlation (DIC) hardware setup.	262
6.29 Load end–shortening curves for the stub column tests.	264
6.30 The deformed 50×50×1, 50×50×2, 50×50×3, 50×50×4 and 50×50×5 SHS stub columns, from left to right.	264
6.31 Normalised ultimate axial resistance varying with local slenderness. . .	266
6.32 EN 1993-1-4 (2015) and CSM compression resistance prediction comparison.	266

List of Tables

3.1	Summary of the tested CHS cross-sections.	94
3.2	Chemical composition as stated in the mill certificates.	94
3.3	CHS tensile coupon crosshead separation rates.	100
3.4	Summary of the tensile coupon material properties.	105
3.5	Summary of the measured geometric properties and test results for the stub columns.	112
3.6	Summary of the stub column material properties.	112
3.7	The number of CHS stub column test results used in the development of the various aspects of the CSM.	114
3.8	The number of CHS four-point bending test results used in the develop- ment of the various aspects of the CSM.	114
3.9	Summary of the coefficients for the CSM material model.	125
3.10	CSM and Eurocode compression resistance prediction comparison for $\bar{\lambda}_c \leq 0.3$	132
3.11	CSM and Eurocode compression resistance prediction comparison for $\bar{\lambda}_c \leq 0.3$	132
3.12	CSM and Eurocode bending resistance prediction comparison for $\bar{\lambda}_c \leq 0.3$.	133
3.13	CSM and Eurocode bending resistance prediction comparison for $\bar{\lambda}_c \leq 0.3$.	133
4.1	Measured geometric properties and test results of the austenitic 106×3 CHS pin-ended columns.	148
4.2	Measured geometric properties and test results of the austenitic 104×2 CHS pin-ended columns.	148

4.3	Measured geometric properties and test results of the duplex 88.9×2.6 CHS pin-ended columns.	148
4.4	Measured geometric properties and test results of the ferritic 80×1.5 CHS pin-ended columns.	149
4.5	Measured geometric properties and test results of the ferritic 101.6×1.5 CHS pin-ended columns.	149
4.6	$N_{u,FE}/N_{u,exp}$ for the 106×3 CHS pin-ended columns.	161
4.7	$N_{u,FE}/N_{u,exp}$ for the 104×2 CHS Finnish pin-ended columns.	161
4.8	$N_{u,FE}/N_{u,exp}$ for the 104×2 CHS Swedish pin-ended columns.	161
4.9	$N_{u,FE}/N_{u,exp}$ for the 88.9×2.6 CHS pin-ended columns.	162
4.10	$N_{u,FE}/N_{u,exp}$ for the 80×1.5 CHS pin-ended columns.	162
4.11	$N_{u,FE}/N_{u,exp}$ for the 101×1.5 CHS pin-ended columns.	162
4.12	$N_{u,FE}/N_{u,exp}$ summary.	163
4.13	$\omega_{u,FE}/\omega_{u,exp}$ for the 88.9×2.6 CHS pin-ended columns.	163
4.14	$\omega_{u,FE}/\omega_{u,exp}$ for the 80×1.5 CHS pin-ended columns.	164
4.15	$\omega_{u,FE}/\omega_{u,exp}$ summary.	164
4.16	Summary of the concentrically loaded pin-ended parametric study specimens.	168
4.17	Comparison between models of full and half cross-section (with symmetrical boundary conditions).	170
4.18	Comparisons of the existing design capacity predictions with the experimental results for class 1–3 cross-sections.	180
4.19	Summary of the statistical analysis results for the EN 1993-1-4 (2015) flexural buckling curve with $\alpha = 0.49$ and $\bar{\lambda}_0 = 0.4$	183
4.20	Summary of the statistical analysis results for the Theofanous et al. (2009b) flexural buckling curve with $\alpha = 0.49$ and $\bar{\lambda}_0 = 0.05$	183
4.21	Summary of the statistical analysis results for the new proposed flexural buckling curve with $\alpha = 0.49$ and $\bar{\lambda}_0 = 0.2$	183

5.1	Summary of the geometric properties and test results for the short 80×1.5 CHS beam-column specimens.	193
5.2	Summary of the geometric properties and test results for the short 101.6×1.5 CHS beam-column specimens.	193
5.3	Summary of the geometric properties and test results for the long 80×1.5 CHS beam-column specimens.	198
5.4	Summary of the geometric properties and test results for the long 101.6×1.5 CHS beam-column specimens.	198
5.5	$N_{u,FE}/N_{u,exp}$ for the 80×1.5 CHS combined axial loading and bending moment columns.	202
5.6	$N_{u,FE}/N_{u,exp}$ for the 101.6×1.5 CHS combined axial loading and bending moment columns.	202
5.7	$N_{u,FE}/N_{u,exp}$ summary for the combined axial loading and bending moment columns.	202
5.8	$N_{u,FE}/N_{u,exp}$ for the 80×1.5 CHS beam-columns.	203
5.9	$N_{u,FE}/N_{u,exp}$ for the 101.6×1.5 CHS beam-columns.	203
5.10	$N_{u,FE}/N_{u,exp}$ summary for the beam-columns.	203
5.11	$\omega_{u,FE}/\omega_{u,exp}$ for the 80×1.5 CHS combined axial loading and bending moment columns.	204
5.12	$\omega_{u,FE}/\omega_{u,exp}$ for the 101.6×1.5 CHS combined axial loading and bending moment columns.	204
5.13	$\omega_{u,FE}/\omega_{u,exp}$ summary for the combined axial loading and bending moment columns.	205
5.14	$\omega_{u,FE}/\omega_{u,exp}$ for the 80×1.5 CHS beam-columns.	205
5.15	$\omega_{u,FE}/\omega_{u,exp}$ for the 101.6×1.5 CHS beam-columns.	205
5.16	$\omega_{u,FE}/\omega_{u,exp}$ summary for the beam-columns.	205
5.17	Summary of the eccentricity loaded pin-ended parametric study specimens.	207
5.18	Proposed stainless steel CHS interaction curve coefficients.	217

5.19 Comparison of the ultimate and predicted combined axial load and bending strength.	220
5.20 Comparison of the ultimate and predicted beam-column strengths for the new and existing test specimens.	221
5.21 Comparison of the ultimate and predicted beam-column strengths for the full dataset.	221
5.22 Summary of the statistical analysis results for the EN 1993-1-4 (2015) stainless steel CHS beam-column design approach.	227
5.23 Summary of the statistical analysis results for the new proposed stainless steel CHS beam-column design approach.	227
6.1 Chemical composition from the manufacturer's datasheet.	231
6.2 Summary of the adopted processing parameters.	236
6.3 Summary of the PH1 measured tensile coupon material properties. . .	241
6.4 Summary of the 316L measured tensile coupon material properties. . .	242
6.5 Summary of typical PH1 and 316L material properties.	243
6.6 Summary of the 316L compressive coupon measured material properties.	254
6.7 Summary of the mean geometric properties of the stub columns. . . .	260
6.8 Summary of stub column tests results and comparisons with existing design methods.	263

Notation

A	Austenitic stainless steel
A	Cross-sectional area
A_{eff}	Effective cross-sectional area
A_f	Tensile coupon area reduction at fracture
A_Y	Integration area
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
AM	Additive manufacturing
AS	Australian Standards
ASCE	American Society of Civil Engineers
B	Bending
B	Outer breadth
b	Average ratio of the experimental to model resistance
BAAM	Big area additive manufacturing
BIM	Building information modelling
C	Carbon or compression
c	R_p strain offset or cross-section element width
C_1	CSM limiting strain coefficient
C_2	CSM material model coefficient for E_{sh}
C_3	CSM material model coefficient for ε_u
C_4	CSM material model coefficient for ε_u
CECS	China Association for Engineering Construction Standardisation

CEN	Comité Européen de Normalisation (European Committee for Standardization)
CHS	Circular hollow section
COV	Coefficient of variation
Cr	Chromium
CSM	Continuous strength method
Cu	Copper
D	Duplex stainless steel
D	Outer CHS diameter
D_L	Loading point vertical displacement
D_M	Midspan vertical displacement
$D_{\max}$	Maximum outer CHS diameter
D_{meas}	Measured outer CHS diameter
$D_{\min}$	Minimum outer CHS diameter
D_{nom}	Nominal outer CHS diameter
D_1	Interaction curve coefficient
D_2	Interaction curve coefficient
D_3	Interaction curve coefficient
DED	Directed energy deposition
DIC	Digital image correlation
DIN	Deutsches Institut für Normung (German Institute for Standardisation)
DMLS	Direct metal laser sintering
DSM	Direct strength method
E	Young's modulus (also known as the modulus of elasticity)
e	Ratio of 0.2% proof stress to Young's modulus = $\sigma_{0.2}/E$
E_c	Compressive Young's modulus
$\dot{e}_{L_c}$	Estimated strain rate over the parallel length
e_{LVDT}	Average LVDT end shortening
e_N	Neutral axis shift

E_{sh}	Strain hardening modulus
E_{stub}	Stub column Young's modulus
e_{True}	True end shortening
e_0	Initial loading eccentricity
$E_{0,\text{LVDT}}$	Young's modulus from LVDT readings
$e_{0,\text{max}}$	Maximum initial loading eccentricity
$e_{0,\text{min}}$	Minimum initial loading eccentricity
$E_{0,\text{True}}$	Young's modulus from strain gauge readings
$E_{0.2}$	Tangent stiffness at 0.2% proof stress
EBM	Electron beam melting
ECAM	Electrochemical additive manufacturing
EHS	Elliptical hollow section
EPSRC	Engineering and Physical Sciences Research Council
F	Ferritic stainless steel or Finland
f_{csm}	CSM limiting material stress
f_u	Ultimate tensile stress
f_y	Material yield stress
f_1	Stress for $D/2$
f_2	Stress for $y \leq Y$
FE	Finite element
Fe	Iron
FIN	Finland
GDP	Gross domestic product
GMAW	Gas metal arc welding
GMNIA	Geometrically and materially nonlinear analyses with imperfections
H	Outer height
I	Second moment of area
ICL	Imperial College London
ILT	Fraunhofer Institute for Laser Technology

K	Ramberg–Osgood constant
k	Beam-column interaction factor or coefficient of proportionality
k_{csm}	Proposed beam-column interaction factor
$k_{\text{d,n}}$	Fractile factor
L	Low carbon content
L	Stub column length or effective length
L_{c}	Coupon parallel length
L_{o}	Original coupon gauge length
L_2	Length of central region between loading points
LENS	Laser engineered net shaping
LUT	Lappeenranta University of Technology
LVDT	Linear variable differential transformer
M	Bending moment
m	Strain hardening exponent
M_{csm}	CSM bending resistance
M_{EC}	Eurocode resistance prediction
M_{Ed}	Design ultimate bending moment
M_{el}	Elastic moment resistance
$M_{\text{N,Rd}}$	Design plastic moment resistance reduced due to the applied axial force
M_{pl}	Plastic moment resistance
M_{R}	Predicted pure bending member strength
$M_{\text{R,csm}}$	CSM resistance reduced due to the applied axial force
M_{Rd}	Pure bending design resistance
M_{u}	Ultimate bending moment
$M_{\text{u,pred}}$	Predicted ultimate bending resistance
Mn	Manganese
Mo	Molybdenum
N	Nitrogen
N	Axial load

n	Ramberg–Osgood strain hardening exponent, the dataset size or axial compressive load level
n_c	Compressive Ramberg–Osgood strain hardening exponent
$N_{b,Rd}$	Flexural buckling resistance
N_{csm}	CSM axial compressive resistance
n_{csm}	Ratio of the compressive design load to the CSM resistance
N_{EC}	Eurocode resistance prediction
N_{Ed}	Design applied axial force
$N_{pl,Rd}$	Design plastic resistance of the cross-section
N_R	Predicted pure compression member strength
N_{Rd}	Pure compression design resistance
n_{stub}	Stub column Ramberg–Osgood strain hardening exponent
N_u	Ultimate axial load
$N_{u,csm}$	CSM axial compressive resistance
$N_{u,EC3}$	EN 1993-1-4 (2015) axial compressive resistance
$N_{u,exp}$	Ultimate experimental compressive capacity
$N_{u,FE}$	Ultimate finite element model compressive capacity
$N_{u,full}$	Ultimate load from the full cross-section model
$N_{u,half}$	Ultimate load from the half cross-section model
$N_{u,pred}$	Predicted ultimate compressive resistance
N_y	Compressive yield load
$n'_{0.2,u}$	Strain hardening exponent passing through $\sigma_{0.2}$ and σ_u
$n'_{0.2,1.0}$	Strain hardening exponent passing through $\sigma_{0.2}$ and $\sigma_{1.0}$
Nb	Niobium
Ni	Nickel
NZS	New Zealand Standards
O	Out-of-roundness tolerance
P	Phosphorus
PBF	Powder bed fusion

r	Radius of curvature
r_e	Average outer corner radius
$r_{e,nom}$	Nominal outer corner radius
r_i	Average inner corner radius
$r_{i,nom}$	Nominal inner corner radius
R_p	Proof stress corresponding to the strain offset c
R–O	Ramberg–Osgood
RHS	Rectangular hollow section
S	Sulphur or Sweden
S_o	Cross-sectional area of the necked region
SABS	South African Bureau of Standards
SANS	South African National Standard
SCI	The Steel Construction Institute
SHS	Square hollow section
Si	Silicon
SLM	Selective laser melting
SLS	Selective laser sintering
SWE	Sweden
t	Wall thickness
t_{meas}	Measured wall thickness
t_{nom}	Nominal wall thickness
UPC	Universitat Politècnica de Catalunya
v_c	Crosshead separation rate
V_{fy}	Coefficient of variation of the yield strength
$V_{geometry}$	Coefficient of variation of the geometry of the cross-section
V_δ	Coefficient of variation of the tests relative to the resistance model
W_{eff}	Effective section modulus
W_{el}	Elastic section modulus
W_{pl}	Plastic section modulus

W_w	Introduced modulus
$W_{w,r}$	Introduced modulus for a circle
Y	Distance to first yield
y	Distance from the elastic neutral axis
$y_{\max}$	Distance from the elastic neutral axis to the extreme fibre
α	Imperfection factor or Rasmussen and Rondal (2000) η coefficient
β	Rasmussen and Rondal (2000) η coefficient
β_w	Section modulus factor
γ_M	Partial safety factor
γ_{M0}	Partial safety factor for the resistance at cross-section level to excessive yielding
γ_{M1}	Partial safety factor for member level resistance to instability
γ_{M2}	Partial safety factor for the resistance of cross-sections in tension to fracture and for connection resistances
δ_{Platen}	End platen deformation
δ_u	End shortening at the ultimate load
ε	Engineering strain or EN 1993-1-4 (2015) material parameter
ε_{cr}	Elastic buckling strain
ε_{csm}	CSM strain at ultimate load
ε_f	Fracture strain over the marked gauge length
ε_{lb}	Local buckling strain at ultimate load
$\varepsilon_{\max}$	Maximum strain
$\varepsilon_{\min}$	Minimum strain
ε_{nom}	Nominal engineering stress
$\varepsilon_{\text{plastic}}$	Plastic strain
$\varepsilon_{\text{true}}$	True strain
$\varepsilon_{t0.2}$	Total strain at $\sigma_{0.2}$
$\varepsilon_{t1.0}$	Total strain at $\sigma_{1.0}$
ε_u	Strain at the ultimate tensile stress

ε_y	Yield strain
η	Imperfection parameter
θ	Axial load bending moment angle parameter or the angle measured from the horizontal building plane to the longitudinal axis of the coupon
κ	Curvature
κ_{el}	Curvature at the elastic moment
κ_u	Curvature at the ultimate moment
$\bar{\lambda}$	Global slenderness
$\bar{\lambda}_c$	Local cross-section slenderness
$\bar{\lambda}_{max}$	Maximum global slenderness
$\bar{\lambda}_{min}$	Minimum global slenderness
$\bar{\lambda}_p$	Local cross-section slenderness
$\bar{\lambda}_0$	Limiting global slenderness
λ_0	Rasmussen and Rondal (2000) η coefficient
λ_1	Rasmussen and Rondal (2000) η coefficient
ν	Poisson's ratio
σ	Applied engineering stress
σ_b	Bending stress
σ_c	Compressive stress
σ_{cr}	Elastic critical buckling stress
σ_{eff}	Effective buckling stress
σ_{nom}	Nominal engineering stress
σ_{true}	True stress
σ_u	Ultimate tensile stress
$\sigma_{0.2}$	0.2% proof stress
$\sigma_{0.2,c}$	Compressive 0.2% proof stress
$\sigma_{0.2,stub}$	Stub column 0.2% proof stress
$\sigma_{1.0}$	1.0% proof stress
$\sigma_{1.0,c}$	Compressive 1.0% proof stress

ϕ	Operational parameter, the end rotation or the angle between the long face of the cross-section and the building plane
ϕ_u	Average end rotation at the ultimate load
χ	Non-dimensional buckling reduction factor
ψ	Ratio of difference and sum of the compression and bending stresses or the angle between the recoating direction and the longitudinal axis of the coupon
ω	Mid-height lateral deflection
ω_{full}	Mid-height lateral deformation at the ultimate load from the full cross-section model
ω_{half}	Mid-height lateral deformation at the ultimate load from the half cross-section model
ω_u	Mid-height lateral deflection at the ultimate load
$\omega_{u,exp}$	Mid-height deflection at the ultimate experimental compressive capacity
$\omega_{u,FE}$	Mid-height deflection at the ultimate finite element model compressive capacity
ω_0	Initial imperfection amplitude

List of Publications

The following is a list of publications, in chronological order, that were produced while undertaking the work described in this thesis.

Gardner, L., Law, K. and Buchanan, C. (2014). Unified slenderness limits for structural steel circular hollow sections. *Romanian Journal of Technical Sciences, Applied Mechanics*, 59(1–2):153–163.

Buchanan, C., Gardner, L. and Liew, A. (2015). The continuous strength method for circular hollow sections. In Batista, E., Vellasco, P. and Lima, L., editors, *Proceedings of the 15th International Symposium on Tubular Structures*, pages 621–628, Rio de Janeiro. Balkema.

Buchanan, C. (2015). A new design method for circular hollow sections. *Innovation & Research Focus*.

Buchanan, C., Gardner, L. and Liew, A. (2016). The continuous strength method for the design of circular hollow sections. *Journal of Constructional Steel Research*, 118:207–216.

Buchanan, C., Real, E. and Gardner, L. (2016). Beam-column behaviour of ferritic stainless steel CHS members. In Schafer, B., editor, *Proceedings of the 7th International Conference on Coupled Instabilities in Metal Structures*, Baltimore, Maryland.

Buchanan, C., Real, E. and Gardner, L. (2016). Testing and design of stainless steel CHS columns. In Young, B., editor, *Proceedings of the 8th International Conference on Steel and Aluminium Structures*, Hong Kong.

Buchanan, C., Matilainen, V-P., Salminen, A. and Gardner, L. (2016). Testing of additive manufactured metallic coupons and cross-sections. In Young, B., editor, *Proceedings of the 8th International Conference on Steel and Aluminium Structures*, Hong Kong.

Buchanan, C., Matilainen, V-P., Salminen, A. and Gardner, L. (2017). Structural performance of additive manufactured metallic material and cross-sections. *Journal of Constructional Steel Research*. (accepted)

Buchanan, C., Gardner, L., Zhao, O. and Real, E. (2017). Design of stainless steel CHS beam-columns. In Heidarpour, A. and Zhao, X. L., editors, *Proceedings of the 16th International Symposium on Tubular Structures*, Melbourne. (submitted)

Chapter 1

Introduction

1.1 General background

Stainless steel gained its name from its initial use in the early 1910s as a ‘rustless steel’ used for cutlery. Stainless steel is still synonymous with cutlery today, in addition to kitchen utensils and appliances, jewellery, automotive exhaust systems and trim, surgical and medical instruments and implants, and in construction for architectural cladding and increasingly as a structural material due to its mechanical and corrosion resisting properties. The term ‘stainless steel’ is applied to iron alloys with corrosion resisting properties and containing a minimum of 10.5% chromium. A thin chromium-rich oxide film forms on the surface in the presence of oxygen which provides the corrosion resistance (Gardner, 2005).

Within construction, austenitic, duplex and ferritic grades are common types of stainless steel used for structural elements and are included in the European stainless steel structural design code EN 1993-1-4 (2015). Austenitic grades are the most prevalent and have a typical chromium content of 17 to 18% and a nickel content of 8 to 11%. Duplex grades offer generally better properties, with higher corrosion resistance, good wear resistance and higher strength, but also have a greater initial cost with a typical chromium content of 22 to 23% and a nickel content of 4 to 5%. Ferritic grades have

a lower initial cost due to their reduced chromium and nickel content, typically 11 to 17% and less than 2.5% respectively, albeit at the expense of corrosion resistance (International Stainless Steel Forum, 2007).

Historically, stainless steel has been used in construction predominantly as a cladding material, a famous example being the Chrysler Building in New York, completed in 1930, with its upper 88 m clad in stainless steel, as shown in Figure 1.1 (Baddoo, 2013). Stainless steel is also commonly used for prominent outdoor public artwork and sculptures such as the 2006 Cloud Gate sculpture in Chicago, Illinois, depicted in Figure 1.2. The use of stainless steel as a structural material in construction is anticipated to increase in the future due to its favourable properties, compared with the more traditionally used carbon steel. Stainless steel has high stiffness, strength and ductility, good fire properties and high recyclability and inherent corrosion resistance, which can lead to a lower whole life cost than an equivalent carbon steel structure. The 2006 United States Air Force Memorial in Arlington, Virginia (Figure 1.3) is an example of a stainless steel being used structurally and for its aesthetics. The complete structure is a stiffened stainless steel shell, with the lower two thirds being a concrete–stainless steel composite system (Powell and McCafferty, 2012).

Circular hollow sections (CHS) are a common form of structural element that have been used for almost 200 years (Dutta, 2002). They are popular with architects and structural engineers due to their aesthetics and numerous structural benefits over other open and closed cross-sections, such as a high torsional resistance, the ability to be filled with concrete to act as a composite member, reduced drag loading in a fluid, good bi-axial bending resistance and reduced maintenance requirements, with a smaller exposed external area. With an expected increase in stainless steel use and CHS being a popular structural cross-section, it is important that design rules for stainless steel CHS are safe and efficient. Two recent examples of the structural use of stainless steel CHS include the Helix Bridge in Singapore, which features a double helix stainless steel CHS tubular truss



Figure 1.1: The Chrysler Building, New York with stainless steel cladding.



Figure 1.2: The stainless steel Cloud Gate sculpture in Millennium Park, Chicago, Illinois.

shown in Figure 1.4, and the structural steelwork supporting the facade of the entrance lobby in the Cordis Hotel, Mong Kok, Hong Kong (Figure 1.5); in both cases stainless steel CHS has been used due to its corrosion resistance in a humid, tropical climate and for its aesthetics.

Design rules require experimental data for the calibration of resistance functions and to compare the capacity predictions. A major focus of this study has been the undertaking of an experimental programme to produce new stainless steel CHS structural data for stub columns and long columns, both concentrically and eccentrically loaded. This new experimental data, along with additional finite element modelling, has been used to evaluate the provisions of the existing European stainless steel design standard EN 1993-1-4 (2015) and to propose improvements at the cross-sectional level for pure compression and bending, and at the member level for flexural buckling and beam-column resistance.

Additive manufacturing, a common example of which is 3D printing, has been the subject



Figure 1.3: The United States Air Force Memorial in Arlington, Virginia.



Figure 1.4: The Helix Bridge, Singapore with a stainless steel CHS truss.

of tremendous societal excitement in recent years. Metallic additive manufacturing is becoming more prevalent in high added value fields, such as the aerospace, automotive, defence and medical industries, due to its numerous advantages over traditional manufacturing techniques. It is starting to be considered as a viable manufacturing technique in construction, although to date, the majority of additive manufacturing constructional research has focussed on cementitious material and plastic construction, with limited focus on metallic structures. It is apparent that there is a requirement to examine the structural properties of metallic additively manufactured elements and to evaluate the applicability of current metallic design standards to structural elements produced using this novel manufacturing technique.



Figure 1.5: Stainless steel CHS in the entrance lobby of the Cordis Hotel, Mong Kok, Hong Kong.

1.2 Research aims

The primary aim of this study is to ensure that structural design methods for stainless steel CHS are safe and efficient, with a secondary aim to investigate the potential for metallic additive manufacturing in construction and the applicability of existing design approaches to this new manufacturing technique. These aims will be achieved through the collection of existing experimental data from the literature, the production of new test data where required, additional finite element modelling to provide a more comprehensive dataset, appraisal of existing design guidance and the development of new more appropriate design rules where required.

1.3 Outline of the thesis

A review of literature related to this thesis is presented in Chapter 2. This chapter provides an overview of the key topics on CHS, stainless steel, finite element modelling and additive manufacturing, with additional relevant literature introduced, where appropriate, throughout the thesis.

The experimental programme, encompassing tensile coupon tests, stub column tests, flexural buckling tests, combined loading cross-section tests and beam-column member tests on austenitic, duplex and ferritic stainless steel CHS, is introduced in Chapter 3. Tensile coupon properties are presented and additional stainless steel CHS stub column results are produced, which are relevant to the structural tests presented throughout the thesis. The chapter then focuses on the local cross-section behaviour of CHS under pure compression and bending, and a new more accurate and less scattered design approach, the continuous strength method (CSM), is extended to cover structural carbon steel, stainless steel and aluminium CHS.

Chapter 4 outlines an extensive experimental programme to produce new stainless steel

CHS flexural buckling data, covering a wide range of local and global slenderness values and austenitic, duplex and ferritic grades of stainless steel. Validated finite element models are generated and a parametric study is undertaken to produce additional data with which to evaluate the existing flexural buckling curve proposals and to develop a new safe, but still efficient, stainless steel CHS flexural buckling curve.

Chapter 5 presents new ferritic CHS combined loading tests at both the cross-section and member level and compares the results with the current European EN 1993-1-4 (2015) design guidance and the CSM. Finite element models of the experiments are developed, validated against the tests, and then a parametric study is undertaken to produce additional stainless steel CHS beam-column member data with which to propose more efficient interaction factors.

Chapter 6 discusses the manufacture, preparation and testing of additive manufactured stainless steel tensile and compressive coupons and square hollow section (SHS) stub columns. The measured tensile and compressive material properties are compared against existing additive manufactured material property tests from the literature. The results of the SHS stub column tests are compared with data from conventionally produced stainless steel SHS and the suitability of the current European EN 1993-1-4 (2015) design approach and the CSM design method for additively manufactured metallic cross-sections is considered.

The major conclusions from this study are presented in Chapter 7, along with topics suitable for additional research in the future.

Chapter 2

Literature review

The literature relevant to this thesis is reviewed in this chapter. The importance of circular hollow sections (CHS) as structural elements is first examined. Focus is then placed on the physical properties of stainless steel, comparisons with carbon steel, stainless steel use in construction and previous experimental investigations into stainless steel hollow sections. A review of stainless steel material models is presented along with the development of existing design standards. The history and reasoning behind the continuous strength method (CSM) is then discussed along with recent developments. Finite element modelling is discussed with a focus on residual stresses, element type, geometric imperfections and the solver used. Finally, a review of metallic additive manufacturing is presented, including the key manufacturing techniques, advantages and disadvantages over traditional manufacturing techniques, an overview of existing metallic research and its current use in construction.

2.1 Circular hollow sections

The first steel circular hollow sections (CHS) were produced by Cornelius Whitehouse in 1825 in Birmingham by heating a steel strip, bending the heated strip into a tube shape and then pressing the ends together to form the tube. Further refinements came in Germany in 1845 with continuously welded tubes and later in the 1880s with seamless CHS (Dutta, 2002).



Figure 2.1: The Forth Bridge, Queensferry, Scotland with CHS main compression members.

The Forth Bridge, completed in 1890, is an example of a structure from the 19th century that makes significant use of CHS structural members with the main cantilever compression members being CHS (Figure 2.1), formed from riveted rolled plates due to fabrication limits at the time (Wardenier et al., 2011). Two more recent examples of high profile CHS use include the two-hinged supporting arch for the Skybridge of the Petronas Towers, completed in 1996 in Kuala Lumpur, Malaysia (Thornton et al., 1997), as shown in Figure 2.2, which utilises hollow CHS with internal tuned mass dampers, and the main compression ring of the London Eye, opened in 2000, which is a triangular CHS lattice (Mann, 2012), as seen in Figure 2.3.

The rise of high-tech architecture, where the structure of a building is not hidden away but becomes part of the architectural form, as apparent from the more recent CHS examples, has led to increased use of CHS due to their aesthetic appearance. However



Figure 2.2: The Petronas Towers, Kuala Lumpur with a two-hinged CHS arch supporting the Skybridge.

their popularity is not simply due to their aesthetics, CHS have a number of structural property advantages over other open and closed cross-sections. When comparing open and closed cross-sections with a similar mass per length (i.e. the same cross-sectional area) CHS are considered to have a good overall strength to weight ratio as the minor axis bending resistance is high compared to other cross-sections, making it ideally suited to scenarios with bi-axial or multi-axis bending. Structurally, CHS are the most efficient cross-sectional shape for providing torsional resistance due to the high geometrical torsional constant that the cross-section possesses, which can be up to ten times greater than open cross-sections with a similar cross-sectional area. Hollow sections have less drag loading in a fluid compared with open sections, which reduces the design loading and allows a smaller cross-section to be chosen. The internal void within CHS can be put to other uses, e.g. mechanical and electrical systems could utilise the space, air and water could be circulated around a structure or it could be filled with concrete which significantly enhances its strength while also allowing a longer fire resistance



Figure 2.3: The London Eye with a CHS triangle lattice compression rim.

time without external fire protection. Hollow sections benefit from having reduced maintenance requirements with a smaller external surface area exposed to corrosive environments, which it has been estimated can halve the maintenance costs, and in particular for CHS there are no edges which can make it easier to clean, paint and generally maintain. Bolted connections with hollow sections can require additional effort as hand access holes are normally required or external connection plates are used; welded connections offer an alternative and are commonplace throughout tubular construction (Dutta, 2002).

It should be noted that CHS use is not purely restricted to high profile structures; they are also common in structures that are not open to the public or that are temporary and thus aesthetics are not an important consideration in their design. The roof of the London Underground Stratford Market Depot features a CHS truss (shown in Figure 2.4), allowing long spans between the supporting columns. Another example is the use of CHS props to prevent the Royal Albert Dock walls from collapsing during the reconstruction of the Connaught Tunnel as part of the Crossrail project, as visible in Figure 2.5.

CHS can either come as a seamless or welded tube, with differing outer diameter and wall thickness limitations and separate manufacturing techniques. Seamless CHS can have an outer diameter up to 1450 mm and a wall thickness up to 270 mm thick and are formed by piercing a heated ingot with a piercing mandrel to create a hollow shell, upon which additional processes are undertaken to turn it into the final desired dimensions. Welded CHS can have a larger outer diameter, up to 2000 mm, which is typically only limited by the logistics of final transportation, although the wall thickness is thinner, at up to 40 mm. Welded CHS are similarly formed in a two stage process — a flat sheet is formed into a tube with a slit (the forming process can cause the slit to be longitudinal or spiral along the tube length) and then the slit is welded, forming the final tube (Dutta, 2002). SHS and RHS are typically formed through hot or cold-working from CHS.



Figure 2.4: Stratford Market Depot, Newham, London with a CHS truss supported roof.



Figure 2.5: The Connaught Tunnel cofferdam, Newham, London with CHS props supporting the dock walls.

2.2 Stainless steel

2.2.1 The ‘discovery’ of stainless steel

Stainless steel was developed by a range of scientists and metallurgists in both Europe and the US simultaneously, and is commonly attributed to having been ‘discovered’ between 1912 to 1913. In 1821 Pierre Berthier, a French metallurgist, noticed the corrosion resistance of iron–chromium alloys, although it was only between 1904 and 1911 that researchers, again in France, continued to further investigate iron–chromium alloys. At the same time in Germany, it was seen that as the chromium content increased there was a corresponding improvement in corrosion resistance, particularly when the chromium content was at least 10.5%. In the UK in 1912, Brearley was developing metallic alloys for gun barrels and observed some discarded samples were not corroding and later produced a steel alloy with 12.8% chromium and termed this ‘rustless steel’ (Baddoo, 2013). This martensitic alloy was initially used for cutlery, where it developed the more commonplace name ‘stainless steel’ (Gardner, 2005). Today, the term stainless steel is given to iron alloys with corrosion resisting properties and containing a minimum chromium content of 10.5%. Between 1912 to 1914, Maurer and Strauss in Germany were also working on iron–chromium alloys and developed austenitic stainless steel. In the US in 1911, a martensitic stainless steel was developed by Haynes, while ferritic stainless steel was produced by Becket and Dantsizen between 1911 to 1914 (Baddoo, 2013). Today stainless steel can be cast, forged, hot-rolled, cold-formed and even additively manufactured (Mann, 1993; Baddoo, 2013).

2.2.2 Types of stainless steel

As stainless steel refers simply to iron alloys with a minimum chromium content there is a tremendously wide range of stainless steel grades available that vary in metallurgical composition and structure, corrosion resistance and mechanical properties. In addition to iron (Fe) and chromium (Cr) additional alloying elements can include carbon (C),

nickel (Ni), manganese (Mn), molybdenum (Mo), copper (Cu), silicon (Si), sulphur (S), phosphorus (P) and nitrogen (N) (Gardner, 2005). There are five basic types of stainless steel classified by their microstructure: austenitic, ferritic, duplex (two phase austenitic–ferritic microstructure), martensitic and precipitation hardening grades. In construction, austenitic, duplex and ferritic grades are the most commonly used types of stainless steel and are included in international design standards. Austenitic grades are the most prevalent and have a typical chromium content of 17 to 18%, a nickel content of 8 to 11%, an austenitic microstructure and good corrosion resistance, forming and fabrication properties. Ferritic grades have a lower initial cost due to their reduced chromium and nickel content, typically 11 to 17% and 0 to 2.5% respectively, albeit at the expense of corrosion resistance (International Stainless Steel Forum, 2007). Ferritic grades tend to have a slightly higher proof strength than austenitic grades and are less prone to stress corrosion cracking. They were historically used in construction for plumbing pipework inside buildings as they had moderate corrosion resistance and poor fabrication properties, although higher performance grades have been developed that can be formed and joined using typical methods such as welding (Baddoo, 2008). Duplex grades offer generally better properties, combining austenitic and ferritic properties through a mixed austenitic–ferritic microstructure (Mann, 1993), with higher corrosion resistance, good wear resistance and higher strength than austenitics, ferritics and many carbon steel grades (although the higher strength does reduce formability (Baddoo, 2008)), however they also have a greater initial cost with a typical chromium content of 22 to 23% and a nickel content of 4 to 5%. Cheaper lean duplex grades have also been developed which have a reduced nickel content but still offer high strength, although with slightly reduced corrosion resistance (Baddoo, 2013). Martensitic grades are less common in construction and are not included in any major design standards; they are used where high abrasion resistance is required (Mann, 1993). Precipitation hardening grades, which can reach very high strengths through heat treatment, are common in the aerospace industry, although are occasionally used in construction for heavy duty connections, tie-bolts and reinforcing bars (Baddoo and Burgan, 2001).

There is a wide range of naming conventions for stainless steel. EN 1993-1-4 (2015) refers to stainless steel grades using a number that is assigned to a specific chemical composition provided in EN 10088-1 (2014a); alternative naming systems include the German (DIN) and US (AISI) systems (Gardner, 2005). As an example, a common austenitic grade can be known using the steel designation number EN 1.4432 from EN 10088-1 (2014a), using the German (DIN) system and also the steel designation name from EN 10088-1 (2014a) as X2CrNiMo17-12-3 and using the US (AISI) system as 316L, with the 'L' signifying a lower carbon content. It should also be noted that a number of grades are patented by stainless steel manufacturers (Mann, 1993; British Standards Institution, 2014a).

2.2.3 Corrosion resistance

A very thin 5 nm chromium-rich oxide film forms on the surface of stainless steel, in the presence of oxygen, which provides the corrosion resistance. This passively forming layer requires a clean surface to form and can be easily damaged by scratching or cutting, although it quickly reforms if oxygen is available (Mann, 1993; Gardner, 2005; Baddoo, 2013). Additional corrosion resistance is also provided by nickel and molybdenum (Gardner, 2005). Despite its name, stainless steel can still stain, pit and tarnish, even in low corrosive environments.

Corrosion is the naturally occurring deterioration of a material due to a chemical reaction with its environment. In the 1970s it was estimated that corrosion in industrialised countries cost 3 to 4% of the gross domestic product (GDP) through the direct costs of replacing damaged components (Baddoo, 2013). There are multiple types of corrosion which can affect metallic materials — general (uniform), abrasive, pitting, crevice, stress, bimetallic (galvanic), and intergranular corrosion (Mann, 1993; Gardner, 2005). General (uniform) corrosion is where the metallic material corrodes over the entire

exposed surface, which should not occur with stainless steels. Abrasive corrosion is the gradual degradation of a surface due to mechanical wear, while pitting corrosion is caused by the local breakdown of the passive layer leading to localised pits forming on the surface (International Stainless Steel Forum, 2010). Crevice corrosion is the result of the movement of oxidants being restricted leading to corrosion within crevices. Stress corrosion cracking can be problematic for some austenitic stainless steel grades and can occur with tensile stresses and chloride ions, even at temperatures as low as 25°C, leading to a brittle failure (Page and Anchor, 1988; BSSA). Bimetallic (galvanic) corrosion occurs when two dissimilar metals are in electrical contact and bridged by an electrolyte which results in a current flowing and the less noble metal corroding. Intergranular corrosion (also known as weld decay) can occur in austenitic grades with prolonged exposure to heat in the 450 to 850°C range, and can lead to carbon diffusing and leaving a lower chromium content near grain boundaries, with implications for the reforming of the passive layer (The Steel Construction Institute, 2006).

The stainless steel grades are chosen based on their cost, corrosion resistance, mechanical properties (strength, ductility, heat resistance), weldability, quality of the surface finish, long-term appearance and the availability of the required cross-sections and shapes (Mann, 1993). The corrosion resistance depends upon the chemical composition and the exposed environment. Increased corrosion resistance typically leads to higher material costs; duplex grades are more expensive and corrosion resistant than austenitic grades, while ferritic grades are the cheapest and have the lowest corrosion resistance. Grade selection is often ultimately a trade-off between corrosion resistance and cost.

2.2.4 Comparisons with carbon steel

Stainless steel is highly durable allowing it to fulfil a service life in excess of 100 years with minimal maintenance, if the correct grade is chosen (Gardner, 2005). The initial cost of stainless steel can be up to four times greater than carbon steel, predominantly

due to the cost of the alloying elements and the low production rate. When taking into account the corrosion resistance, savings on the application of coatings and reduced maintenance, the whole life cost can be less for stainless steel than for carbon steel (Mann, 1993). Improved fire resistance and, with duplex grades, greater strength and the potential for reduced section sizes and lower weight also offer further cost savings (Gardner, 2005).

Stainless steel has generally similar mechanical properties to carbon steel, although there are some differences. Stainless steel has a rounded stress–strain response with strain hardening occurring after first yield, whereas carbon steel has a plastic yield plateau before the onset of strain hardening. Consequently, there is no precise yield point for stainless steel and an equivalent that is defined using an offset 0.2% proof stress $\sigma_{0.2}$ is typically used. Stainless steel, in the annealed condition, has a lower minimum 0.2% proof stress $\sigma_{0.2}$ than the equivalent carbon steel yield stress f_y , the minimum nominal strength is 175 N/mm² from EN 1993-1-4 (2015) whereas for carbon steel from EN 1993-1-1 (2014) the minimum is 215 N/mm² and typically higher strength carbon steel grades are used. The minimum stainless steel proof stresses provided by manufacturers are often exceeded by a large margin; in one study the average measured proof stress was 40% larger than the specified minimum and the lowest measured value was 26% greater than the minimum value (Mann, 1993; Baddoo, 2008). Design based on the minimum quoted strengths, which also ignore potential strength benefits from strain hardening, would therefore be conservative, but not efficient or economical, as additional available strength is being neglected. Some types of stainless steel can undergo significant strain hardening, as apparent from the ratio between the ultimate stress σ_u and 0.2% proof stress $\sigma_{0.2}$, which for austenitic stainless steel grades can be up to 2.4, while the equivalent for typical carbon steels can be between 1.4 to 2.1. The rounded stress–strain response leads to a varying Young's modulus E with nonlinear deflections and greater deflections than similar carbon steel structures (Mann, 1993). The magnitude of the Young's modulus E is also slightly lower for stainless steel than

carbon steel, typically 200,000 N/mm² compared with 210,000 N/mm². Stainless steel has higher ductility than carbon steel which leads to excellent impact loading resistance, although the amount of ductility depends upon the composition, heat treatment and the level of cold-working (Gardner, 2005). The material properties of stainless steel are both slightly anisotropic and non-symmetric; i.e. the properties vary depending upon direction and whether loaded in tension and compression. Both stainless steel and carbon steel undergo strength enhancement through cold-working, which results in a higher yield and ultimate stress although with reduced ductility (Mann, 1993; Rossi et al., 2013).

Stainless steel has been observed to have improved elevated temperature behaviour compared to carbon steel; austenitic stainless steels have better strength retention above 550°C and better stiffness retention at all temperatures (Gardner and Baddoo, 2006; Baddoo, 2013). However the coefficient of thermal expansion is up to 50% greater for stainless steel which can induce additional loads into other parts of a structure. The residual stresses and fatigue resistance of stainless steel is similar to carbon steel. Imperfections are reduced in stainless steel structural elements as there are typically tighter controls on appearance due to the increased prominence of stainless steel structural elements (Gardner, 2005). Austenitic grades are non-magnetic, while duplex and ferritic grades are magnetic (Mann, 1993; Baddoo, 2008).

Stainless steel is completely recyclable and can be indefinitely recycled, and hence has a high residual scrap value and incentive to be recycled (Gardner, 2005). It is estimated that 92% of industrial, building and infrastructure stainless steel is captured at the end of its life. Typical European and US stainless steel contains between 75 to 90% recycled material, although this is limited by the availability of scrap stainless steel due to its long service life (Baddoo, 2013).

In summary stainless steel has high durability, stiffness, strength and ductility, more favourable elevated temperature properties, is highly recyclable and corrosion resistant

and can often have a lower whole life cost compared with carbon steel.

2.2.5 Stainless steel in construction

Stainless steel has historically been a more specialist metallic material in construction, predominantly due to its high initial cost, though improved design guidance and a general trend towards more sustainable construction is leading to its increased use. It was estimated that in 2006 four million tonnes, 14% of all stainless steel consumed, was used in construction (Baddoo, 2008). Stainless steel has been used in roofing and for facades since the 1920s. In 1925 stainless steel circumferential bands were used to restrain the dome of St Paul's Cathedral in London (Figure 2.6) against spreading and stainless steel reinforcing bars and tie rods were used within columns supporting the dome. A major potential use for stainless steel is in renovation, repair and maintenance, with 45% of all construction expenditure in this area (Baddoo, 2008). The Chrysler Building in New York, completed in 1930, has its top 88 m clad in stainless steel (shown in Figure 1.1 in Chapter 1) and has only needed to be cleaned twice. The Gateway Arch in St Louis, Missouri, US, completed in 1965, was the first large structural application of stainless steel with 804 tonnes of stainless steel used. Stainless steel has been used predominantly for cladding; in London, examples include the 1963 Millbank Tower, the 1986 Lloyd's building, which used 30,000 m² of stainless steel for cladding and pipework and is shown in Figure 2.7, and the 1991 One Canada Square building (Figure 2.8). Stainless steel is also frequently used for bridge construction due to their frequent location in corrosive marine environments. In 2005, the first stainless steel road bridge in Europe was opened in Minorca, Spain, while other examples include the 2009 Stonecutters Bridge in Hong Kong, shown in Figure 2.9, and the 2010 Helix Bridge in Singapore (Figure 1.4 in Chapter 1), which both used duplex stainless steel due to its higher strength (Vejrum et al., 2006; Baddoo, 2013). Stainless steel is increasingly used for structural and architectural elements particularly in the offshore, water treatment, nuclear, biomass, chemical, pharmaceutical and foodstuff industries (Baddoo, 2013). The high cost of stainless steel requires efficient material use and therefore accurate

design rules. The development of stainless steel design rules are covered in Section 2.3.

Stainless steel sheet, plate, bar, reinforcing bar, tubular and proprietary fixings products are widely available in austenitic, duplex and ferritic grades. Austenitic hot rolled and welded I-sections, angles and tees are also commonplace, while special orders are necessary for other types of stainless steel (Baddoo, 2013). The stocked cross-sections tend to be smaller in size and limited to 6 m in length, although additional cross-sections and shapes can easily be formed through cold-forming, extrusion and laser welding (Mann, 1993). There is also a choice of available surface finishes — plain, textured, matt and polished, which are affected by the manufacturing techniques. Arguments have been made that due to the more limited supply of pre-manufactured cross-sections and shapes, and based on the high cost of the raw material, that a shift away from standardised shapes can be justified as the increased labour costs would be offset by reduced material costs (Mann, 1993). Stainless steel is also commonly used for fixings, particularly for use with wood and masonry as these tend to be corrosive to other metallic materials by absorbing moisture and corrosive chemicals, and are often located in hard to access locations that are difficult or impossible to replace (Baddoo, 2013).

Stainless steel requires additional care during fabrication processes, with the main requirement that it is kept separate from carbon steel to prevent surface contamination that could spoil the surface finish. Dedicated stainless steel tools should be used and for prominent structural elements additional surface cleaning and wrapping of finished products during transportation may be required to ensure a good surface finish (Mann, 1993). Cutting and drilling stainless steel can also present some challenges due to the work hardening properties, which can require tools to be sharpened or replaced more frequently than typically with carbon steel — this was encountered when machining the tensile coupons in Section 3.4.2. All stainless steel grades can be welded, including being welded to carbon steel (the welded end plates in Chapter 5 are carbon steel), although some additional care is required to ensure that the beneficial properties of stainless steel



Figure 2.6: The dome of St Paul's Cathedral, London with stainless steel circumferential bands.



Figure 2.7: The Lloyd's Building, London with 30,000 m² of stainless steel cladding and pipework.

are not reduced through welding. As discussed previously, prolonged heating in the 450 to 850°C temperature range can cause weld decay through chromium depletion and, to address this, lower carbon 'L grades' were developed, although an appropriate welding technique that minimises the time spent within the critical range can also prevent this.

It is important to note that historically there have been a number of fatal incidents involving stainless steel structural failures, predominately around swimming pools. In 1985 a concrete ceiling, suspended by austenitic stainless steel rods, collapsed onto a swimming pool in Uster, Switzerland killing 13 people, with the failure attributed to stress-corrosion cracking which caused catastrophic brittle failure in the normally ductile rods. The building had been in use for 13 years and it was previously thought that such corrosion could not occur below 50°C (Page and Anchor, 1988; BSSA). In 2011 a similar incident occurred in the Netherlands, with a speaker assembly falling into a swimming pool and killing a child, and this was again attributed to stress-corrosion



Figure 2.8: Stainless steel cladding on One Canada Square, London.



Figure 2.9: The Stonecutters Bridge, Hong Kong with a composite stainless steel skin on the upper portion of the main towers.

cracking, this time in a stainless steel bolt (Heselmans and Vermeij, 2013). It is clear that an alternative choice of stainless steel grade would have prevented these failures, although for the Switzerland incident, stress-corrosion cracking was not believed to occur with the present environmental conditions.

2.2.6 Existing stainless steel hollow section experiments

Safe and efficient design rules require experimental results for the calibration of resistance functions and to evaluate the capacity predictions. The historical lack of stainless steel hollow section experimental data has led significant effort being devoted to structural testing in recent years with stub column tests, bending tests, long column tests, eccentrically-loaded stub column tests and beam-column tests having been undertaken on austenitic, duplex and ferritic circular hollow sections (CHS), square hollow sections (SHS), rectangular hollow sections (RHS) and elliptical hollow sections (EHS), which are outlined herein.

Stainless steel CHS stub column tests have been undertaken on austenitic grades by Rasmussen and Hancock (1993a), Talja (1997), Rasmussen (2000), Young and Hartono (2002), Kuwamura (2003), Gardner and Nethercot (2004a), Lam and Gardner (2008), Uy et al. (2011) and Zhao et al. (2015a), on duplex grades by Bardi and Kyriakides (2006), Paquette and Kyriakides (2006) and Lam and Gardner (2008) and on ferritic grades by Stangenberg (2000). The austenitic CHS stub column tests conducted by Talja (1997) are also reported by Burgan et al. (2000).

SHS and RHS stub column tests have been carried out on austenitic grades by Rasmussen and Hancock (1993a), Talja and Salmi (1995), Young and Liu (2003), Liu and Young (2003), Kuwamura (2003), Gardner and Nethercot (2004a), Young and Lui (2005), Gardner et al. (2006), Lam and Gardner (2008) and Yuan et al. (2014), on duplex grades by Young and Lui (2005), Lam and Gardner (2008), Theofanous and Gardner (2009),

Huang and Young (2012) and Yuan et al. (2014) and on ferritic grades by Afshan and Gardner (2013a), Bock et al. (2015a), Arrayago and Real (2015), Zhao et al. (2015e) and Zhao et al. (2016b).

Stainless steel CHS beam tests have been carried out on austenitic grades by Rasmussen and Hancock (1993b) and a combined testing programme is reported by Talja (1997), Burgan et al. (2000) and Kiymaz (2005). The only duplex stainless steel CHS beam tests are jointly reported by Burgan et al. (2000) and Kiymaz (2005).

Rasmussen and Hancock (1993b), Talja and Salmi (1995), Mirambell and Real (2000), Gardner and Nethercot (2004b), Zhou and Young (2005), Gardner et al. (2006) and Theofanous et al. (2014) have undertaken experimental testing on austenitic SHS and RHS simply supported beams and Mirambell and Real (2000) and Theofanous et al. (2014) on continuous beams. Simply supported duplex SHS and RHS stainless steel beams have been tested by Zhou and Young (2005), Theofanous and Gardner (2010) and Huang and Young (2013a). Afshan and Gardner (2013a), Bock et al. (2015a), Zhao et al. (2015e), Arrayago and Real (2016) and Zhao et al. (2016b) have tested simply supported ferritic SHS and RHS beams, while Arrayago and Real (2016) has also undertaken experimental testing on continuous ferritic SHS and RHS beams.

Historically only austenitic CHS long column tests have been undertaken, with pin-ended conditions by Rasmussen and Hancock (1993a) and Zhao et al. (2016c), jointly reported by Talja (1997) and Burgan et al. (2000), and with fixed ends by Young and Hartono (2002).

SHS and RHS flexural buckling tests have been carried out on pin-ended austenitic columns by Rasmussen and Hancock (1993a), Talja and Salmi (1995), Gardner and Nethercot (2004b) and Gardner et al. (2006) and on fixed ended columns by Young and Liu (2003) and Liu and Young (2003). For duplex stainless steel grades pin-ended

flexural buckling SHS and RHS tests have been performed by Theofanous and Gardner (2009) and Huang and Young (2013b) and on ferritic grades by Afshan and Gardner (2013a), Zhao et al. (2016b) and Arrayago et al. (2016).

Combined axial load and bending moment tests have been undertaken at the cross-section level by Zhao et al. (2015a) for austenitic CHS. For SHS and RHS at the local level combined loading tests have only been carried out on ferritic stainless steel grades by Zhao et al. (2015e) and Arrayago and Real (2015).

At the member level, CHS beam-column tests have been carried out by Rasmussen and Hancock (1993a) and Zhao et al. (2016c), and jointly reported by Talja (1997) and Burgan et al. (2000), for austenitic stainless steel grades. SHS and RHS beam-column tests have been undertaken by Rasmussen and Hancock (1993a) and Talja and Salmi (1995) on austenitic grades, by Lui et al. (2014) and Huang and Young (2014) on duplex grades and by Zhao et al. (2016b) and Arrayago et al. (2016) for ferritic grades.

Testing on stainless steel EHS is much more limited, with tests focussing on austenitic grades — stub column tests have been reported by Theofanous et al. (2009b), bending tests by Theofanous et al. (2009a) and long column tests by Theofanous et al. (2009b).

2.2.7 Stainless steel material models

In the literature, significant effort has been directed towards developing material models that can replicate the stress–strain response of stainless steel, allowing the full material behaviour to be represented using a small number of standardised parameters.

Ramberg and Osgood (1943) developed an expression for the stress–strain response of aluminium alloys, which also have a rounded stress–strain response, and is reproduced as

$$\varepsilon = \frac{\sigma}{E} + K \left(\frac{\sigma}{E} \right)^n, \quad (2.1)$$

where ε and σ are the engineering strain and stress, E is the Young's modulus and K and n are constants; the expression is a summation of the elastic and plastic strains. Ramberg and Osgood (1943) noted the applicability of Equation 2.1 to stainless steels, and this was subsequently modified by Hill (1944) to give

$$\varepsilon = \frac{\sigma}{E} + c \left(\frac{\sigma}{R_p} \right)^n, \quad (2.2)$$

where R_p is a proof stress and c is the corresponding offset plastic strain. This model became popular as the key parameters have a physical significance (the Young's modulus E , a proof stress R_p corresponding to a strain c and the strain hardening exponent n , which defines the curvature of the stress–strain response — a higher n value denotes a sharper curve) and it can reproduce a smooth continuous stress–strain response (Ashraf et al., 2006b). The 0.2% proof stress is typically adopted, resulting in the traditional form of the Ramberg–Osgood expression,

$$\varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{\sigma_{0.2}} \right)^n. \quad (2.3)$$

The strain hardening exponent n is typically determined using either the 0.05% (Rasmussen and Hancock, 1993b) or 0.01% (Rasmussen, 2003) proof stress point and the 0.2% proof stress. This expression has been noted to provide a very good fit to experimental stress–strain responses up to the 0.2% proof stress, but beyond can significantly overpredict the stresses (Mirambell and Real, 2000; Ashraf et al., 2006b).

Mirambell and Real (2000) proposed a two-stage material model, where the traditional Ramberg–Osgood approach is adopted up to the 0.2% proof stress and then a modified formula is used for stresses above this point,

$$\varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{\sigma_{0.2}} \right)^n \quad \text{for } \sigma \leq \sigma_{0.2} \quad (2.4)$$

and

$$\varepsilon = \frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \varepsilon_u \left(\frac{\sigma - \sigma_{0.2}}{\sigma_u - \sigma_{0.2}} \right)^m + \varepsilon_{t0.2} \text{ for } \sigma < \sigma_{0.2} \leq \sigma_u, \quad (2.5)$$

where $E_{0.2}$ is the tangent stiffness at the change over point $\sigma_{0.2}$, ε_u is the plastic strain at the ultimate tensile strength, σ_u is the ultimate tensile strength, m is an additional strain hardening exponent and $\varepsilon_{t0.2}$ is the total strain at the 0.2% proof stress. Equation 2.5 is essentially a translated expression that ensures continuity of the gradient and stress-strain magnitude. The tangent stiffness $E_{0.2}$ can be calculated from

$$E_{0.2} = \frac{\sigma_{0.2}E}{\sigma_{0.2} + 0.002nE}. \quad (2.6)$$

Mirambell and Real (2000) recommended determining the strain hardening exponent n from the 0.05% and 0.2% proof stresses, based on previous work by Rasmussen and Hancock (1993b). Rasmussen (2003) further developed the model, and an expression for the additional strain hardening exponent m was presented based on experimental data,

$$m = 1 + 3.5 \frac{\sigma_{0.2}}{\sigma_u}. \quad (2.7)$$

Equations were also proposed to predict the ultimate tensile strength σ_u ,

$$\frac{\sigma_{0.2}}{\sigma_u} = \frac{0.2 + 185 (\sigma_{0.2}/E_0)}{1 - 0.0375 (n - 5)}, \quad (2.8)$$

and the corresponding strain at the ultimate tensile strength ε_u ,

$$\varepsilon_u = 1 - \frac{\sigma_{0.2}}{\sigma_u}. \quad (2.9)$$

Rasmussen (2003) recommended calculating the strain hardening exponent n using the 0.01% and 0.2% proof stresses. This model has been included in Annex C of EN 1993-1-4 (2015) for modelling the material behaviour of stainless steel.

Gardner and Nethercot (2004a) noted two drawbacks with the two-stage model approach — firstly using the strain at the ultimate tensile strength ε_u results in a greater deviation between the measured and predicted stress–strain responses for $\sigma_{0.2} \leq \sigma \leq \sigma_u$ than if a lower strain value were used for the second part of the curve. Secondly, the two-stage models are only suitable for tensile stress–strain behaviour since, without necking, there is no ultimate stress σ_u in compression. Gardner and Nethercot (2004a) proposed using the 1.0% proof stress $\sigma_{1.0}$ instead of the ultimate stress σ_u , leading to an updated form of the second stage,

$$\varepsilon = \frac{(\sigma - \sigma_{0.2})}{E_{0.2}} + \left(0.008 - \frac{\sigma_{1.0} - \sigma_{0.2}}{E_{0.2}}\right) \left(\frac{\sigma - \sigma_{0.2}}{\sigma_{1.0} - \sigma_{0.2}}\right)^{n'_{0.2,1.0}} + \varepsilon_{t0.2} \text{ for } \sigma_{0.2} < \sigma \leq \sigma_u \quad (2.10)$$

where $n'_{0.2,1.0}$ is a strain hardening exponent for a curve passing through the 0.2% and 1.0% proof stresses. Using this notation the m exponent in Equation 2.5 could alternatively be termed $n'_{0.2,u}$. This second stage was further refined by Ashraf et al. (2006b), leading to

$$\varepsilon = \frac{(\sigma - \sigma_{0.2})}{E_{0.2}} + \left(\varepsilon_{t1.0} - \varepsilon_{t0.2} - \frac{\sigma_{1.0} - \sigma_{0.2}}{E_{0.2}}\right) \left(\frac{\sigma - \sigma_{0.2}}{\sigma_{1.0} - \sigma_{0.2}}\right)^{n'_{0.2,1.0}} + \varepsilon_{t0.2} \text{ for } \sigma_{0.2} < \sigma \leq \sigma_u \quad (2.11)$$

where $\varepsilon_{t1.0}$ is the total strain at $\sigma_{1.0}$.

Cold-formed stainless steel structural sections can undergo significant cold-working during the manufacturing process — with the sheet material being coiled for storage, uncoiled for section forming and then the actual section forming process (Afshan et al., 2013). Karren (1967) investigated the strength enhancement in the corner regions of cold-formed carbon steel sections. Work on predicting the strength enhancement for stainless steel has been undertaken by Coetzee et al. (1990), Van den Berg and Van der Merwe (1992), Gardner and Nethercot (2004a), Ashraf et al. (2005), Cruise and Gardner (2008b), Rossi (2008) and Rossi et al. (2013).

2.3 Existing design codes and methods

2.3.1 Stainless steel design standards

Before the advent of dedicated stainless steel design guidance structural engineers had to develop their own design rules or simply avoid utilising stainless steel in a structural capacity. The 1965 Gateway Arch in St Louis, Missouri, US kickstarted research into the structural response of stainless steel in the 1960s with the first stainless steel design specification being published by the American Iron and Steel Institute (AISI) in 1968 (American Iron and Steel Institute, 1968; Baddoo, 2008). A revised specification was published by the AISI in 1974 based on an experimental programme undertaken at Cornell University (Johnson and Winter, 1966; American Iron and Steel Institute, 1974). This was superseded in 1991 by an American Society of Civil Engineers (ASCE) standard and again in 2002 (American Society of Civil Engineers, 2002). The 2002 standard was focussed on the design of cold-formed stainless steel members, while the American Institute of Steel Construction (AISC) produced a design specification for hot-rolled and welded open sections and rectangular and round hollow sections (American Institute of Steel Construction, 2013).

In 1994, Euro Inox published a design manual based on stainless steel design rules developed from a Steel Construction Institute (SCI) project (Euro Inox, 1993). The design manual led to the ENV 1993-1-4 (1996) draft standard published by the European Standards organisation CEN, and this was later converted into a full European Standard EN 1993-1-4 (2006) and subsequently updated as EN 1993-1-4 (2015). EN 1993-1-4 (2006) was the first stainless steel design standard in many European countries and supplements, extends and modifies the provisions of EN 1993-1-1 (2014), EN 1993-1-3 (2010), EN 1993-1-5 (2006b) and EN 1993-1-8 (2010) for carbon steel to apply to stainless steel. EN 1993-1-4 (2015) must be used alongside these carbon steel standards

and covers hot-rolled, cold-formed and welded austenitic, duplex and ferritic stainless steel structural members. Consequently the stainless steel design approach is based on the carbon steel approach with modifications where required to fit the experimental data better (Gardner, 2005); as an example the concept of cross-section classification is adopted in EN 1993-1-4 (2015) but with differing classification limits to deal with the different stress–strain characteristics. Baddoo (2013) noted the need to find a balance between complicated design rules that take full advantage of the differing stress–strain behaviour of stainless steel with the relative ease for structural engineers of adopting the carbon steel approach with minor modifications.

AS/NZS 4673 (2001) in Australia and New Zealand and SANS 10162-4/SABS 0162-4 (1997) in South Africa were developed based on the American stainless steel standards (Baddoo, 2008). The 1995 Japanese stainless steel standard for welded fabricated sections was published and a subsequent design guide for lightweight stainless steel members was later published in 2005 (Gardner, 2005; Baddoo, 2008). The Chinese CECS 410 (2015) stainless steel technical specification has also recently been released.

2.3.2 The continuous strength method (CSM)

The continuous strength method (CSM) has been developed in recent years to reflect the observed characteristics of metallic structural elements better. A simplified elastic, perfectly plastic material model is adopted in EN 1993-1-4 (2015); this model can be justified for carbon steel, with a defined yield point and plastic yield plateau, but for stainless steel with a nonlinear, but smoother, stress–strain response, and significant strain hardening, such a material model is less appropriate. Current design codes also use the concept of cross-section classification to separate cross-sections into discrete classes depending upon their susceptibility to local buckling. In the European metallic design standards (Eurocode 3 EN 1993-1-1 (2014) and BS 5950-1 (2000) for structural carbon steel, Eurocode 3 EN 1993-1-4 (2015) for stainless steel and Eurocode 9 EN

1999-1-1 (2007) for aluminium) four classes of cross-section are considered, while in the AISC 360 (2010) and AS 4100 (1998) structural carbon steel codes there are three cross-section classes.

Gardner and Nethercot (2004d) argued that with a continuous stainless steel material stress–strain response, a continuous, rather than a discontinuous or discretised, design approach should be adopted to take full advantage of the available material strength. A new design approach was proposed, unnamed at the time, where a single, unique numerical value defines the deformation capacity of a cross-section, replacing cross-section classification, and more accurate material models are utilised. The deformation capacity is turned into a normalised local buckling strain ε_{lb} , and then the material model from Gardner and Nethercot (2004a), reproduced as Equation 2.10 in Section 2.2.7, is inverted to determine the local buckling stress, which is used in place of, and can exceed, the 0.2% proof stress $\sigma_{0.2}$. The bending resistance is calculated using a generalised shape factor that includes the material and geometric characteristics. Expressions were also proposed for the flexural buckling resistance and beam-column resistance. Gardner and Nethercot (2004d) focussed on stainless steel SHS, RHS and CHS, noting that the new design method gave improved predictions compared to traditional methods, and stated that the general approach could be extended to other cross-sections.

Ashraf et al. (2006a) extended this new design approach to look at open stainless steel cross-sections including angles, channels, lipped channels and H sections, in compression. Gardner and Ashraf (2006) applied the design method to other nonlinear metallic materials including aluminium, high strength carbon steels and cold-worked carbon steels, and it was again found to provide more consistent and accurate cross-section resistances than existing provisions. New continuous relationships between local slenderness and deformation capacity were proposed for aluminium, stainless steel and high strength steel. The updated material model developed by Ashraf et al. (2006b), reproduced as Equation 2.11, was adopted. Ashraf et al. (2008b) made a slight

modification to the stainless steel deformation capacity curve and proposed new ways of applying the design method to cover flexural buckling and beam-columns. Lam and Gardner (2008) applied the method to concrete filled stainless steel CHS columns, by modifying the contribution from the steel tube.

The new approach was named the ‘continuous strength method’ (CSM) by Gardner (2008), who proposed a number of changes from the earlier work. Gardner (2008) focussed on structural carbon steel, collecting SHS, RHS and lipped channel stub column data and CHS bending test data. New expressions to determine the local slenderness and deformation capacity were presented, along with a new elastic, linear strain hardening material model that does not require inversion and new integrated bending expressions for I sections and CHS to replace the previous generalised shape factors. Reference is made to the previous work on aluminium, stainless steel and high-strength steel by noting that there are significant benefits for nonlinear material with significant strain hardening characteristics.

Gardner et al. (2011b) focussed again on carbon steel and proposed a new expression linking the local slenderness to the deformation capacity for non-slender I sections, SHS and RHS, which is termed the base curve for the first time. An updated major axis bending resistance expression for an I section or box section, suitable for design use, is provided along with a design approach for applying the CSM to indeterminate structures.

Afshan and Gardner (2013b) proposed a new non-slender base curve for plated cross-sections based on a linear regression fit to collected stainless steel test data. A stainless steel material model suitable for austenitic and duplex grades of stainless steel was also developed, compatible with the approach from Gardner (2008). Bock et al. (2015b) later developed a material model for ferritic stainless steel grades. Yuan et al. (2014) applied the CSM to built up stainless steel I sections, SHS and RHS.

Su et al. (2014b) developed an aluminium material model, again compatible with Gardner (2008), and adopted the non-slender base curve developed by Afshan and Gardner (2013b) and found it to offer improved predictions of the compressive resistance for aluminium stub columns over current design standards. Su et al. (2014a) applied this work to aluminium beams, and again observed more accurate and consistent predictions compared with conventional methods.

Liew and Gardner (2015) adopted the non-slender base curve from Afshan and Gardner (2013b), produced analytical equations for the minor axis bending of I sections and box sections and then developed simplified expressions suitable for use in design. The CSM was then further extended to cover these cross-sections under combined axial load and bending moment.

Zhao et al. (2015d) applied the design expressions from Liew and Gardner (2015) to stainless steel SHS and RHS under combined loading and also applied the existing EN 1993-1-4 (2015) interaction formulae but with the CSM end points, which was called the ‘simplified CSM’ approach, with both found to offer more accurate and consistent predictions of cross-section level resistance compared with current design methods. Zhao et al. (2015b) investigated non-slender stainless steel CHS under combined loading and proposed two interaction curves, building upon the work presented in Chapter 3.

Su et al. (2016) proposed a base curve for slender plated cross-sections by modifying the coefficients of existing strength based design methods to fit collected carbon steel, stainless steel and aluminium plated cross-section data. Foster and Gardner (2016) undertook experiments on carbon steel I sections undergoing lateral torsional buckling and proposed a lateral torsional slenderness limit below which the CSM bending resistance can be attained. Foster et al. (2015) proposed modifications to the hot-rolled and cold-formed carbon steel material model, on the basis of it taking the form of a two-stage piecewise linear material model, while Gardner et al. (2016) developed

a four-stage piecewise linear material model for hot-rolled carbon steel, incorporating both the yield plateau and subsequent strain hardening.

The CSM is included in the AISC Steel Design Guide for Structural Stainless Steel (American Institute of Steel Construction, 2013) and the recent fourth edition of the Design Manual for Structural Stainless Steel (The Steel Construction Institute, 2017).

2.4 Finite element modelling

Physical experimentation often has a high time and monetary cost and therefore it is not economically viable to produce all of the desired data through structural testing alone. Finite element modelling is an essential tool to simulate the behaviour of structural elements, as it can closely replicate reality with significantly reduced cost.

An example of finite element modelling used in structural engineering research is the work by Saab and Nethercot (1991). Two-dimensional steel frames under fire conditions were modelled in a computer program developed by the authors based on a previous finite element analysis framework (El-Zanaty and Murray, 1983). Blachut et al. (1991) and Blachut and Galletly (1995) investigated the buckling of steel hemispheres and used the finite element package ABAQUS to replicate the physical experiments. The failure modes were well replicated, although the measured ultimate experimental capacity normalised by the ultimate load from the numerical simulation varied from 0.56 to 1.21. Yuen (1992) investigated carbon steel I sections under flexure and similarly replicated the experiments numerically using ABAQUS, and then undertook a parametric study. Talja and Salmi (1995) used ABAQUS to model stainless steel SHS and RHS columns under concentric and eccentric loading and under flexural loading. Since the mid 1990s finite element modelling, specifically using ABAQUS, has become commonplace and is now an essential tool for structural engineering researchers.

The finite element modelling approach adopted in this study is outlined in more detail in Section 4.3. Residual stresses, mesh element type and size, geometric imperfections and the solver used are discussed further herein.

2.4.1 Residual stresses

Residual stresses can cause early yielding and loss of stiffness, resulting in a lower ultimate capacity of structural elements, particularly those in which stability is important. Two types of residual stresses can be considered: i) through thickness residual stresses arising from non-uniform plastic deformation during cold-forming and ii) membrane residual stresses from welding (Zhao et al., 2015a). Membrane stresses within cold-formed stainless steel tubes have previously been investigated (Rasmussen and Hancock, 1993a; Cruise and Gardner, 2008a; Jandera et al., 2008; Huang and Young, 2012) and have been observed to be small compared to the cold-forming stresses, and considered safe to neglect. The influence of the bending residual stresses are apparent from coupons that are cut from cold-formed sections, which typically curve longitudinally upon release (see Figure 3.11 in Section 3.4.3). These bending stresses are reintroduced during testing when the coupon becomes straight again under loading and therefore the effect of the through thickness bending residual stresses are considered to be present in the measured material stress–strain response. As common with previous studies the effects of the residual stresses have therefore been neglected due to their small effect or implicit incorporation in the material models used (Gardner, 2002; Ellobody and Young, 2005; Young and Ellobody, 2006; Ellobody and Young, 2007; Zhao et al., 2015a, 2016c).

2.4.2 Mesh elements and size

Shell elements are typically used to model thin-walled structures, such as stainless steel hollow sections. The four-noded doubly curved S4R shell element with reduced

integration, with six degrees of freedom per node, is commonplace and is suitable for finite strain analysis. The S4R shell element has been used in the context of stainless steel research by Gardner and Nethercot (2004c), Ellobody and Young (2005), Young and Ellobody (2006), Ellobody and Young (2007), Kiymaz et al. (2007), Ellobody (2013), Zhao et al. (2015a) and Zhao et al. (2016c) among others for CHS, by Gardner and Nethercot (2004c), Theofanous and Gardner (2009), Huang and Young (2015), Zhao et al. (2015e) and Zhao et al. (2016b) for RHS and SHS, by Theofanous et al. (2009a) and Theofanous et al. (2009b) for elliptical hollow sections (EHS) and by Hassanein and Silvestre (2013) for I sections.

The mesh size influences the accuracy of the numerical results and the computation time; therefore, a mesh size that provides accurate failure loads, deformations and mode shapes compared to those measured in the experiments, but that is still computationally efficient, is desired. Previous CHS studies have adopted uniform mesh dimensions in the circumferential and longitudinal directions and this approach has been adopted in this study following a prior mesh sensitivity study (Zhao et al., 2015a, 2016c).

2.4.3 Geometric imperfections

Geometric imperfections are an inherent feature of metallic structural members and finite element models should include both local and global initial geometric imperfections. It is commonplace to undertake an elastic buckling analysis before the nonlinear analysis and to use a chosen eigenmode as an initial imperfection that simulates the imperfect geometry and does not prevent the observed buckling modes. This approach is used not just to simulate geometric imperfections but also residual stresses, material non-homogeneity and eccentric loading (Ashraf et al., 2006b). Local geometric imperfections occur mainly during fabrication and can affect the development of local buckling, the load level at which plasticity initiates, the ultimate load and the post-ultimate response (Zhao et al., 2015b). It is therefore important to include an appropriate representative local

geometrical imperfection in the numerical models, and typically the lowest local buckling mode shape under the modelled loading condition is applied, with the amplitude of the imperfection is taken as a fraction of the wall thickness t ; although in certain scenarios, such as with mode interaction, higher modes may be more appropriate (Girão Coelho et al., 2013). Hassanein and Silvestre (2013), Su et al. (2014a), Zhao et al. (2015a), Zhao et al. (2015b) and Zhao et al. (2015e) modelled short metallic specimens and only included local imperfections.

Global imperfections are similarly normally included as the lowest global buckling mode shape, with the amplitude of the imperfection specified as a fraction of the member length L . Local and global modes are typically both included in finite element models and are superimposed together. Gardner and Nethercot (2004c), Becque et al. (2008), Theofanous et al. (2009b), Theofanous and Gardner (2009), Huang and Young (2015), Zhao et al. (2016b) and Zhao et al. (2016c) included both local and global imperfections in their numerical models.

It should be noted that this approach is used for modelling convenience, and that modelling initial imperfections as eigenmodes is not necessarily realistic, as highlighted by the choice of imperfection magnitudes which are typically based on achieving a measured ultimate capacity from an experiment, rather than linked to a geometric measurement of the imperfection (Schafer et al., 2010), though the chosen imperfection magnitudes are often linked to manufacturing tolerances.

2.4.4 Solver

Abaqus includes Lanczos and subspace iteration eigenvalue extraction methods. The subspace iteration method can be faster when fewer than 20 eigenmodes are required and has been used in a number of previous studies to extract the buckling mode shapes (Theofanous and Gardner, 2009; Theofanous et al., 2009a; Theofanous and Gardner, 2010). The Lanczos method is typically faster when a larger number of eigenmodes are

required for a system with many degrees of freedom (Dassault Systèmes, 2015a) and has again been used in previous finite element studies (Fratamico and Schafer, 2014; Ozyurt and Wang, 2015). The Lanczos method was used as in some cases a large number of eigenmodes were required to attain both local and global buckling shapes.

Buckling or collapse behaviour, such as that seen in geometrically and materially nonlinear static problems require the structure to release strain energy to remain in equilibrium and have load–displacement responses with negative stiffness. The modified Riks method (Riks, 1972, 1979) can be used to find static equilibrium states during the unstable phase, provided that the response is smooth and that no sudden bifurcations occur. It can be used where the loading is proportional, where the load is governed by a single scalar parameter (the arc length), and solves simultaneously for the load and displacement (Dassault Systèmes, 2015a). The modified Riks method (Riks, 1972, 1979) has successfully been used in previous finite element analysis studies to follow the nonlinear post-ultimate response of metallic structural elements (Gardner and Nethercot, 2004c; Ellobody and Young, 2005; Chan and Gardner, 2008a; Theofanous et al., 2009a; Foster and Gardner, 2013; Su et al., 2014a; Rotter et al., 2014; Hradil and Talja, 2014; Zhao et al., 2015a; Bock et al., 2015b) and is also used in the finite element modelling in this study.

2.5 Additive manufacturing

2.5.1 Introduction

Additive manufacturing is the overarching name for any form of manufacturing where an object is produced in an additive manner, whether this be through the laying down of material, melting and binding of material, selective curing of a liquid or the adhering of existing layers. ‘3D printing’ is a term that generates tremendous excitement in society, although strictly applies only to methods which lay down material, and is itself a type of additive manufacturing. Additive manufacturing first developed in the 1980s through

rapid prototyping techniques, initially through stereolithography, which is the process of solidifying ultraviolet sensitive liquid polymer with a laser (Wong and Hernandez, 2012). Significant advancements have been made since then with a wide range of materials able to be utilised including ceramics, chemicals, composites, concrete, foodstuffs, metallic materials (including aluminium, cobalt-chrome, copper, gold, iron alloys (including stainless steels), magnesium, nickel based alloys, titanium and tungsten), paper, plastics, sandstone, silicones, wax and wood (Wong and Hernandez, 2012; Symes et al., 2012; Niendorf et al., 2013; Yap et al., 2015; Abe et al., 2001; Cesaretti et al., 2014; Sun et al., 2015; Pearce et al., 2010; Lim et al., 2012; Henke and Treml, 2013). The end products are equally varied from controlling chemical reactions, with both the polymer reaction vessel and chemical reagents being produced by the same printer (Symes et al., 2012), customised sports footwear (Wohlers and Caffrey, 2013), building plastic tools on the International Space Station (Snyder et al., 2013), the production of titanium rocket nozzle components (Song et al., 2015) and even personalised chocolate confectionary (Sun et al., 2015).

2.5.2 Metallic additive manufacturing

Metal additive manufacturing is increasingly popular in the aerospace, automotive, defence and medical industries (Cooper et al., 2012; Gu et al., 2012) due to its many advantages over traditional manufacturing techniques, such as casting, fabrication, machining, rolling and stamping. Figure 2.10 shows a test firing of a nickel superalloy additive manufactured engine chamber by SpaceX, where the part could be manufactured in days rather than taking months if it were cast and with higher material strength and reduced material property variability (SpaceX, 2014). Conventionally manufactured objects often have simple geometries to aid manufacturing (Yap et al., 2015) whereas with new production techniques this need not be the case. Additive manufacturing enables automated and repeatable rapid prototyping or small production runs of objects with complex geometries and shapes that would otherwise be time and cost prohibitive, or even impossible with other manufacturing methods. Structural components can

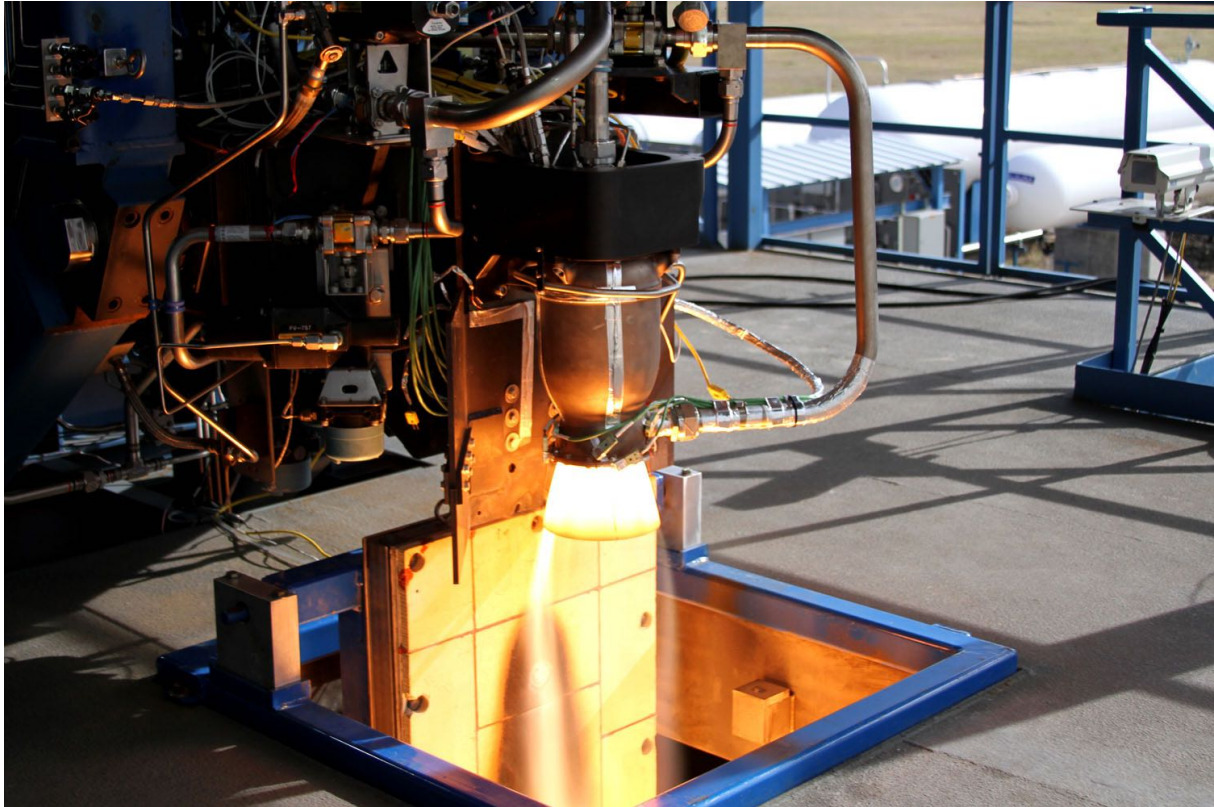


Figure 2.10: The test firing of an AM nickel superalloy engine chamber (SpaceX, 2014).

take highly optimised lightweight forms to carry loads more efficiently, including the possibility of internal stiffening structures and specific weight-saving porosities with parts as thin as 100 microns (Santorinaios et al., 2006). There are also a number of wider benefits including reduced material waste, reduced energy input and rapid incorporation of design changes. Parts can be built essentially anywhere, just-in-time if required and with the same processing parameters, predictably and reliably. Additive manufacturing techniques can even be used to repair worn and damaged metallic components (Yap et al., 2015; Gu et al., 2012). This new manufacturing process has also been found to yield advantageous mechanical properties due to the rapid cooling over conventionally produced metallic materials (Kobryn and Semiatin, 2001).

To date, compared with plastic additive manufacturing, which has benefited from the introduction of low cost production equipment in the late 1990s (Wohlers and Gornet, 2011), equipment suitable for use with metallic materials has been prohibitively expensive (Anzalone et al., 2013). Currently, production time can be excessively lengthy

since, with some building methods, objects are built up using individual layers that are tens of microns thick. Support structures are typically required to restrain the part being manufactured mechanically to the rigid building platform, in order to reduce the distortion arising from the residual stresses (Khalid Rafi et al., 2013), and these need to be later manually removed. The surface finish is different from conventionally produced metallic objects which, for some applications, may necessitate additional finishing processes, such as sanding or bead blasting, and consequently parts may need to be manufactured slightly oversized. Significant design times within computer aided design (CAD) software packages can also be required. The maximum dimensions of a single part are limited by the build envelope of the equipment which, for powder bed fusion, is typically a 250 mm cube, while some direct energy deposition methods allow a single dimension up to 5 m. These size limitations will be overcome in time; Big Area Additive Manufacturing (BAAM) is being developed with the intention to build components as large as aircraft wings that are 30 m in length (Holshouser et al., 2013). While additive manufactured metallic materials exhibit some beneficial mechanical properties over conventionally produced materials, they can also exhibit anisotropy (Kobryn and Semiatin, 2001; Tolosa et al., 2010; Mertens et al., 2014; Kruth et al., 2005; Spierings et al., 2011; Mower and Long, 2016; Luecke and Slotwinski, 2014; Kempen et al., 2012; Meier and Haberland, 2008) which for some materials can be partially remediated through heat treatment (Mower and Long, 2016; Luecke and Slotwinski, 2014; Kempen et al., 2012; Prashanth et al., 2014; Vilaro et al., 2011; Facchini et al., 2010), and high residual stresses (Niendorf et al., 2013; Wu et al., 2014).

ISO/ASTM 52900 (2015) provides an outline of metallic single-step additive manufacturing methods, where the key process categories are: i) directed energy deposition, ii) powder bed fusion and iii) sheet lamination. Directed energy deposition (DED) relies on the selective depositing of melted material, which can be either a laser or electron beam melted powder or a filament/wire material, akin to welding. Powder bed fusion (PBF) is where material within a powder bed is selectively fused together using thermal energy

from a laser or electron beam. The third category is sheet lamination, where individual cut out cross-sections can be laminated together using ultrasound. Electrochemical additive manufacturing (ECAM), not covered by ISO/ASTM 52900 (2015), is a new alternative metallic additive manufacturing process which is being developed that is similar to electroplating, where the part is built up at the atomic level (Sundaram et al., 2015). This process is very much in the early stages of development with the maximum dimension of parts currently around 600 μm , but as it is a non thermal process it can be used to build parts with lower internal residual stresses. Beyond single-step processing, there is multi-step processing where the first step provides the geometric shape and the second provides the material properties, and this process can also involve adhering dissimilar materials together. The proprietary nature of the manufacturing equipment and metallic powders has resulted in a variety of names for similar processes. ISO/ASTM 52900 (2015), and its predecessor ASTM F2792-12a (2013), have attempted to standardise the terminology, but this has only recently been adopted in the literature; for example, PBF includes selective laser melting (SLM), selective laser sintering (SLS), direct metal laser sintering (DMLS) and electron beam melting (EBM) among others (Wong and Hernandez, 2012), while DED includes laser engineered net shaping (LENS).

Over the past twenty years there has been significant research into metallic additive manufacturing processes and applications. Tensile coupon tests have been undertaken on PBF 316L stainless steel to investigate anisotropy arising from different building orientations (Tolosa et al., 2010; Mertens et al., 2014; Spierings et al., 2011; Mower and Long, 2016; Meier and Haberland, 2008), the influence of the laser power (Niendorf et al., 2013), powder particle size (Spierings et al., 2011) and building layer thickness (Spierings et al., 2011; Meier and Haberland, 2008) on the material properties, the fatigue performance (Mower and Long, 2016; Spierings et al., 2013) and residual stresses (Wu et al., 2014; Mercelis and Kruth, 2006). Corrosion resistance has also been assessed and, it has been found that provided full relative density is attained, the resistance is similar to conventionally formed material (Sun et al., 2014). Coupon tests have been

performed on PH1/15-5PH martensitic stainless steel to study the tensile behaviour in different build orientations (Khalid Rafi et al., 2013) and to examine the fatigue behaviour (Khalid Rafi et al., 2013; Spierings et al., 2013). Research has also been carried out into DED 316L (Yadollahi et al., 2015), the processing parameters for PBF (Guan et al., 2013) and DED (Kahlen and Kar, 1999) 304 stainless steel, the anisotropy (Mower and Long, 2016; Kempen et al., 2012) and heat treatment (Kempen et al., 2012; Prashanth et al., 2014) of PBF aluminium alloys, the anisotropy (Kobryn and Semiatin, 2001) and heat treatment (Vilaro et al., 2011; Facchini et al., 2010) of PBF titanium alloys, the mechanical properties of DED titanium alloys (Mertens et al., 2012; Baufeld et al., 2010), the structural integrity of post-processed bronze-nickel alloys (Agarwala et al., 1995) and directionality at differing temperatures of a nickel based superalloy (Rickenbacher et al., 2013). Studies into the structural applications of PBF metallic materials have included examining 316L stainless steel open cellular lattice structures (Santorinaios et al., 2006; Shen et al., 2010; Ushijima et al., 2011) and negative Poisson's ratio structures (Rehme, 2009). The applications of DED MX3D gas metal arc welding (GMAW) in structural engineering have also been explored (Joosten, 2015).

2.5.3 Additive manufacturing in construction

Additive manufacturing offers many potential benefits in construction, such as the ability to produce bespoke individual components, reduced waste, time and material savings, more optimised structural forms and easier integration with building information modelling (BIM) (Lim et al., 2012; Perkins and Skitmore, 2015). Current constructional additive manufacturing research has focussed mainly upon cementitious materials. Contour crafting, which utilises a cement-based paste against a trowel can be used to produce a 200 m² single-storey building within a day (Perkins and Skitmore, 2015; Khoshnevis, 2004), concrete printing involves extrusion of cement mortar which is shown in Figure 2.11 (Lim et al., 2012) and the D-shape process involves producing a material similar to sandstone by combining adhesives and a sand-like material (Cesaretti

et al., 2014; Lim et al., 2012; Perkins and Skitmore, 2015; Tibaut et al., 2014). In 2014, construction started on the 3D Print Canal House in Amsterdam, Netherlands which is an additive manufactured house made by joining individual plastic blocks (Perkins and Skitmore, 2015; Bogue, 2013), a development model is shown in Figure 2.12. Construction has started, again in Amsterdam, on the MX3D bridge, a stainless steel DED GMAW structure that will eventually span an 8 m wide canal (Joosten, 2015). In order to validate the manufacturing process structural testing has been undertaken on DED GMAW built struts as shown in Figure 2.13. Arup recently redesigned a node detail to be built using PBF and found that while currently it would cost roughly three times that of a conventionally produced node, it is expected to be cheaper through manufacturing developments within five years (Galjaard et al., 2014). The progression from a conventionally fabricated lighting node to an optimised additive manufactured node is shown in Figure 2.14, with the final node having a weight reduction of 75% over the original node. It has been estimated that in the future, additive manufacturing technologies may decrease construction costs by 30% through automation and reduced labour requirements (Pegna, 1997). Additive manufacturing techniques have also been proposed to aid in the rapid construction of shelters in disaster hit areas (Khoshnevis, 2004).

2.6 Concluding remarks

In this chapter an overview of key literature on stainless steel research, the continuous strength method (CSM) and additive manufacturing has been presented. The importance of circular hollow sections (CHS) as a structural cross-section and an outline of the key stainless steel properties, comparisons with carbon steel and historical use in construction have been discussed. Previous experimental programmes on stainless steel hollow sections were outlined, along with a review of stainless steel material models and the evolution of existing stainless steel design standards. The development of the continuous



Figure 2.11: A test structure being formed through concrete printing (BBC News, 2016).



Figure 2.12: A plastic canal house development model (Frearson, 2016).



Figure 2.13: An MX3D flexural buckling test specimen (Joosten, 2015).

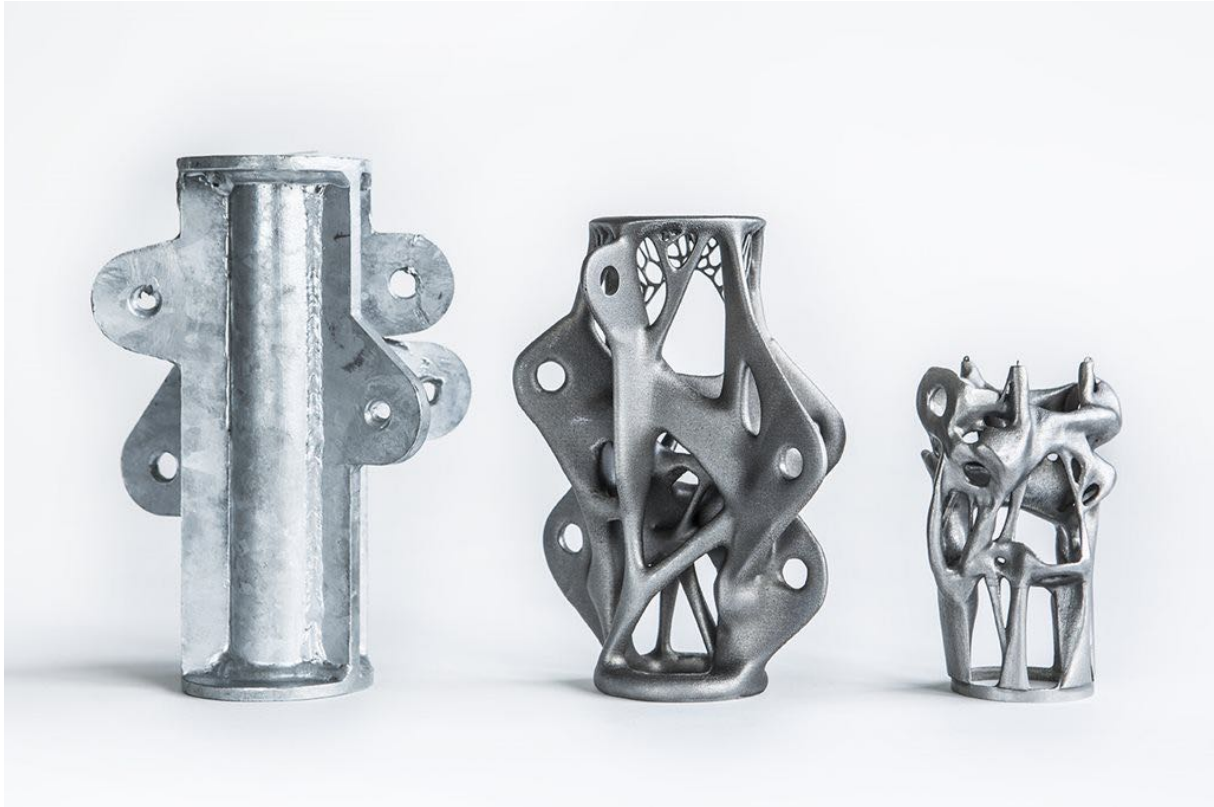


Figure 2.14: The progression from a conventionally fabricated to an optimised, additive manufactured lighting node (Arup, 2015).

strength method is outlined along with a discussion on finite element modelling. A review of additive manufacturing was then presented including its advantages and disadvantages over traditional manufacturing techniques, an overview of existing metallic research and its current use in the construction industry.

Chapter 3

Cross-section behaviour of CHS

3.1 Introduction

CHS are primarily thin-walled structural elements, and therefore local buckling, whether prior or subsequent to material yielding, is a primary consideration in their design. This is the topic of the present chapter. The work in this chapter has been presented previously by Buchanan et al. (2015), Buchanan et al. (2016a), Buchanan et al. (2016c) and Buchanan et al. (2016d).

3.1.1 Traditional CHS design methods

Current design codes use the concept of cross-section classification to separate circular hollow sections into discrete classes depending upon their susceptibility to local buckling. Four classes of cross-section are considered in Eurocode 3 EN 1993-1-1 (2014) and BS 5950-1 (2000) for structural carbon steel, Eurocode 3 EN 1993-1-4 (2015) for stainless steel and Eurocode 9 EN 1999-1-1 (2007) for aluminium. In bending, class 1 cross-sections can reach and maintain their full plastic moment capacity M_{pl} with suitable rotation capacity for plastic design. Class 2 cross-sections are also capable of reaching their full plastic moment capacity but with limited rotation capacity. There is no equivalent to class 2 cross-sections in the AISC 360 (2010) and AS 4100 (1998) structural carbon steel codes. Class 3 cross-sections are unable to reach their plastic

moment capacity due to local buckling and their bending capacity is limited to the elastic moment capacity M_{el} . Class 4 cross-sections experience local buckling before reaching their elastic moment capacity, and are typically referred to as slender. In terms of axial resistance, the class 3 limit separates the non-slender cross-sections that are fully effective in compression (i.e. classes 1–3) from those that fail by local buckling before reaching their yield load (i.e. class 4). These traditional design methods also limit the maximum stress in the cross-section to the yield strength f_y , neglecting the beneficial effects of strain hardening in metallic materials. Experimental results have shown that cross-section classification and limiting the maximum stress to the yield stress can be overly conservative in estimating the resistance of stocky (class 1–3, non-slender) cross-sections (Gardner, 2008). It is therefore apparent that there are structural efficiency improvements to be sought over existing design methods for CHS.

3.1.2 The continuous strength method (CSM)

The continuous strength method (CSM) has been developed in recent years to reflect the observed characteristics of metallic structural elements better. Cross-section classification is replaced with a continuous relationship between cross-section slenderness and deformation capacity (referred to in Section 3.7 as the base curve), reflecting the continuous nature of cross-section capacity varying with local slenderness. A strain hardening material model is also adopted, representing the behaviour seen in material tests, with an increase in strength above the yield strength under plastic deformation.

The CSM has previously been developed for structural carbon steel (Gardner, 2008; Gardner et al., 2011b; Foster, 2014), stainless steel (Afshan and Gardner, 2013b) and aluminium (Su et al., 2014b) plated cross-sections, such as I-sections, square hollow sections (SHS) and rectangular hollow sections (RHS) in compression and bending, and also under combined loading (Liew and Gardner, 2015). The previous work has shown that the CSM predicts enhanced capacities over existing methods; for example, in the

case of stainless steel, average enhancements in compressive and bending resistances of 12% and 19% respectively were found (Afshan and Gardner, 2013b). The natural progression is to extend the application of the CSM to circular hollow sections and the development process is described herein.

3.1.3 Extension of the CSM to CHS design

The process of extending the CSM to cover the design of CHS requires: i) the identification of the yield slenderness limit (i.e. the local slenderness limit below which significant benefit from strain hardening can be derived for non-slender cross-sections); ii) the formulation of the CSM non-slender and slender base curves defining the relationship between cross-section slenderness and deformation capacity; iii) the selection of appropriate material models; and iv) the derivation of resistance functions.

3.2 Cross-section slenderness

The local cross-section slenderness $\bar{\lambda}_c$ is defined in non-dimensional form by

$$\bar{\lambda}_c = \sqrt{\frac{f_y}{\sigma_{cr}}}, \quad (3.1)$$

where f_y is the material yield strength and σ_{cr} is the elastic critical buckling stress — which for a CHS in compression is calculated from

$$\sigma_{cr} = \frac{E}{\sqrt{3(1-\nu^2)}} \frac{2t}{D}, \quad (3.2)$$

where E is the Young's modulus, ν is the Poisson's ratio, D is the outer diameter and t is the wall thickness of the CHS.

Timoshenko and Gere (1961) suggested that the local buckling stress in bending can be taken as 1.4 times that in compression based on experimental results (Donnell, 1934), which effectively makes a cross-section in bending more stocky than the same cross-

section in compression. Gerard and Becker (1957) proposed a factor of 1.3 based upon the findings of Flügge (1932). However, more recent analytical work has showed that the maximum critical stress in bending is equal to the critical compressive stress for practical cylinder lengths (Seide and Weingarten, 1961; Reddy and Calladine, 1978; Rotter et al., 2014). Differences also exist between international design codes in their treatment of compression and bending for CHS. Gardner et al. (2014) noted that EN 1993-1-1 (2014) and EN 1999-1-1 (2007) utilise the same class 3 limits for both compression and bending, in contrast to BS 5950-1 (2000), EN 1993-1-4 (2015), AISC 360 (2010) and AS 4100 (1998) where different limits are used. Utilising different slenderness limits in compression and bending is equivalent to applying a factor to the local buckling stress. Given the findings of the more recent research (Seide and Weingarten, 1961; Reddy and Calladine, 1978; Rotter et al., 2014) and the conservative nature of the choice, the elastic critical buckling stress in bending will be taken to be the same as that in compression (see Equation 3.2) in the present study and within the CSM.

3.3 CHS experimental database

A dataset of 342 experimental results on CHS in compression or bending has been collated from the literature. The dataset includes stub column test results for hot-finished structural carbon steel (Giakoumelis and Lam, 2004; Liew and Xiong, 2010; Nseir, 2015; Ochi and Choo, 1993; Starossek and Falah, 2009), very high strength structural carbon steel (Jiao and Zhao, 2003; Zhao, 2000), cold-formed structural carbon steel (Chen and Ross, 1977; Elchalakani et al., 2002b; Jiao and Zhao, 2003; Johansson and Gylltoft, 2002; Kamba, 1996; Ma et al., 2015; Nseir, 2015; Ochi and Choo, 1993; O'Shea and Bridge, 1997; Prion and Birkemoe, 1992; Ren et al., 2014; Sakino et al., 2004; Schmidt, 1989; Teng and Hu, 2007; Toi et al., 1986; Toi and Ine, 1988; Tutuncu and O'Rourke, 2006; Wei et al., 1995; Xiao et al., 2005; Yu and Teng, 2010; Zhao et al., 2010), austenitic stainless steel (Burgan et al., 2000; Gardner and Nethercot, 2004a; Kuwamura, 2003; Lam and Gardner, 2008; Rasmussen, 2000; Rasmussen and Hancock, 1993a; Talja, 1997; Uy

et al., 2011; Young and Hartono, 2002; Zhao et al., 2015a), duplex stainless steel (Bardi and Kyriakides, 2006; Lam and Gardner, 2008; Paquette and Kyriakides, 2006), ferritic stainless steel (Stangenberg, 2000) and aluminium (Zhou and Young, 2009; Zhu and Young, 2006b), and four-point bending test results for hot-finished structural carbon steel (Gresnigt and van Foeken, 2001; Rondal et al., 1995; Sedlacek et al., 1995; Sherman, 1976), very high strength structural carbon steel (Jiao and Zhao, 2004), cold-formed structural carbon steel (Elchalakani et al., 2002a; Gresnigt and van Foeken, 2001; Guo et al., 2013; Haedir et al., 2009; Jirsa et al., 1972; Kabir and Fawzia, 2014; Ma et al., 2015; Seica et al., 2006; Sherman, 1976), austenitic stainless steel (Burgan et al., 2000; Kiymaz, 2005; Rasmussen and Hancock, 1993b), duplex stainless steel (Kiymaz, 2005) and aluminium (Su et al.; Zhu and Young, 2006a). Note that the very high strength structural carbon steel had a typical yield stress f_y around 1300 MPa (Zhao, 2000).

3.4 Stub column tests

3.4.1 General overview

An experimental programme has been undertaken in the structures laboratories at Imperial College London (ICL) and Universitat Politècnica de Catalunya (UPC), shown in Figures 3.1 and 3.2 respectively, to produce additional stainless steel CHS experimental data to fill gaps in the existing test dataset. Tensile coupon tests, 10 stub column tests, 37 flexural buckling tests, 12 combined loading tests and 10 beam-column tests have been carried out on austenitic (A), duplex (D) and ferritic (F) stainless steel CHS covering cross-section classification classes 1–4. The tensile coupon and stub column tests are described in this section, while the flexural buckling tests are discussed in Section 4.2 and the combined loading and beam-columns tests undertaken are outlined in Section 5.2. The tensile coupon tests and austenitic and duplex experiments were carried out at ICL, while the ferritic experiments were undertaken at UPC.



Figure 3.1: The structures laboratory at Imperial College London.



Figure 3.2: The structures laboratory at Universitat Politècnica de Catalunya.

Two austenitic (A) cross-sections were tested, a 106×3 CHS and a 104×2 CHS, a single duplex (D) cross-section was tested, a 88.9×2.6 CHS, while two ferritic (F) cross-sections were tested, a 80×1.5 CHS and a 101.6×1.5 CHS. The stainless steel grade and type of stainless steel, nominal outer diameter D_{nom} and wall thickness t_{nom} , average measured outer diameter D_{meas} and wall thickness t_{meas} , and the averaged measured diameter to thickness ratio $D_{\text{meas}}/t_{\text{meas}}$ are provided in Table 3.1, along with the EN 1993-1-4 (2015) cross-section class and the local slenderness $\bar{\lambda}_c$ determined using averaged geometric and tensile material properties. The CHS were all produced by cold-forming and longitudinal welding. Prior to cutting it was noted that the 104×2 CHS tubes came from two different sources, both from the country markings (Sweden and Finland) on the tubes and the external finish around the weld, although there was no distinction on the mill certificate. The chemical composition of the specimens, as stated in the mill certificates, are provided in Table 3.2.

It is apparent from Section 2.2.6 in the literature review that there are limited existing ferritic CHS stub column data and these new ferritic stub column tests are designed to provide additional experimental data for the ultimate compressive cross-sectional resistance the end shortening required to attain this resistance level. Stub column tests can also provide useful compressive material properties that are averaged across the cross-section.

3.4.2 Specimen preparation

At the start of the testing programme the uncut lengths of CHS tube were inspected and it was observed that the cross-sections were deformed in certain locations, as apparent in Figure 3.3, and that the lengths had longitudinal global imperfections, as visible from Figure 3.4. EN 1993-1-4 (2015) refers to EN 10219-2 (2006a) for the allowable tolerances on cold formed welded structural sections of non-alloy and fine grain steels, where the ‘out-of-roundness’ tolerance O is 2% for a hollow section with a diameter to

Table 3.1: Summary of the tested CHS cross-sections.

Section	Grade	Type	D_{nom} (mm)	t_{nom} (mm)	D_{meas} (mm)	t_{meas} (mm)	$D_{\text{meas}}/t_{\text{meas}}$	EN 1993-1-4 (2015) compressive class	Local slenderness $\bar{\lambda}_c$
106×3	1.4432	A	106.0	3.0	105.75	2.78	38.1	1	0.20
104×2	1.4307/ 1.4301	A	104.0	2.0	104.04	1.84	56.5	3	0.28
88.9×2.6	1.4462	D	88.9	2.6	88.70	2.36	37.6	4	0.31
80×1.5	1.4512	F	80.0	1.5	80.02	1.34	59.7	3	0.29
101.6×1.5	1.4512	F	101.6	1.5	101.72	1.34	76.2	4	0.31

Table 3.2: Chemical composition as stated in the mill certificates.

Section	Grade	Type	C (%)	Si (%)	S (%)	P (%)	Mn (%)	Cr (%)	Ni (%)	Mo (%)	Ti (%)	N (%)	Co (%)
106×3	1.4432	A	0.020	0.39	0.003	0.029	1.30	16.70	10.60	2.79	-	0.046	-
104×2	1.4307/ 1.4301	A	0.026	0.39	0.001	0.030	1.48	18.10	8.10	-	-	0.049	-
88.9×2.6	1.4462	D	0.015	0.41	0.001	0.018	1.39	22.40	5.75	3.10	-	0.170	-
80×1.5	1.4512	F	0.011	0.46	0.010	0.024	0.27	11.68	0.25	0.04	0.22	0.008	0.01
101.6×1.5	1.4512	F	0.016	0.44	0.001	0.026	0.28	11.55	-	-	0.24	0.012	-



Figure 3.3: The uncut lengths of the 106×3 CHS with ovalised cross-sections.

thickness ratio D/t not exceeding 100, with the roundness being calculated from

$$O = \frac{D_{\max} - D_{\min}}{D_{\text{nom}}} \times 100, \quad (3.3)$$

where $D_{\max}$ and $D_{\min}$ are the maximum and minimum outer diameters measured in the same plane and D_{nom} is the nominal specified outer diameter. Measurements of the outer diameter D were taken, using a digital calliper, at regular intervals along the length (approximately equal to 300 mm) and at varying angles around the cross-section. Where at one position the roundness condition was clearly satisfied, but not at the next, the same exercise would be repeated at a mid-position to determine if more tube could be utilised. Once all of the uncut lengths were measured for roundness, specimens were assigned to lengths that satisfied the ‘out-of-roundness’ requirements from EN 10219-2 (2006a) with an aim to maximise the amount of compliant remaining tube for trial and repeat specimens.

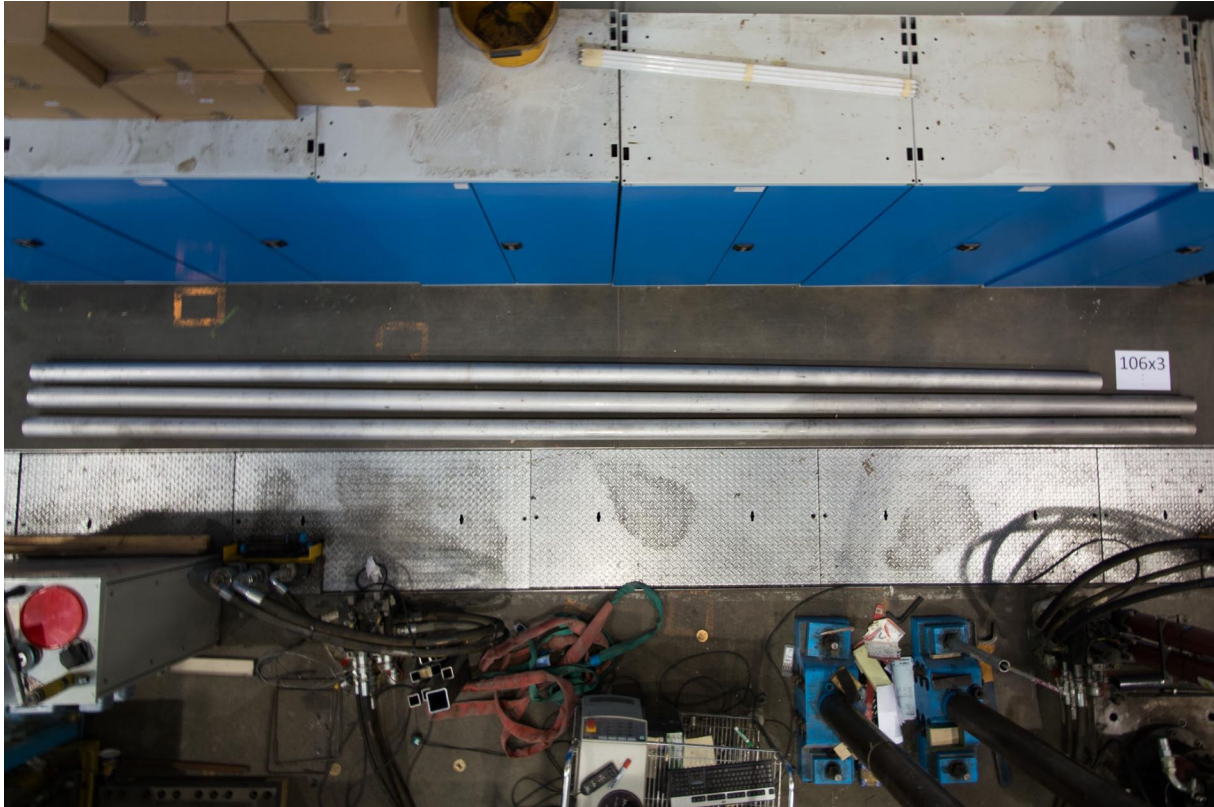


Figure 3.4: The uncut lengths of the 106×3 CHS showing longitudinal deformations.

The austenitic and duplex specimens were cut at ICL using a bandsaw, with a new bandsaw blade intended for use with stainless steel installed prior to cutting. A special round wooden block was machined to fit inside the tubes to reduce cross-section distortion at the clamping and cutting locations and a second rectangular wooden block was inserted between the clamp and outer face of the tube, as shown in Figure 3.5. It was noted post clamping that the wooden inserts could not be moved along the specimen length, even with a clamping force just sufficient to prevent movement of the specimen, highlighting that the blocks were helping to reduce further cross-section distortion. Stands were used to support the unsupported lengths of specimen (the initial uncut length was up to 6000 mm) to prevent damage to the cutting blade and a non-straight cut as the bottom extreme fibres are cut through, while for shorter specimens the unsupported end was held by hand to achieve the same effect.

The shorter specimens ($L \leq 1000$ mm) had their ends machined in a lathe, while a milling machine was used for the longer specimens, to ensure that they were flat and



Figure 3.5: A CHS specimen being cut using a bandsaw, with wooden inserts inside and outside the specimen.

parallel, as shown in Figures 3.6 and 3.7. Bespoke aluminium inserts were machined and placed inside the end of the clamped end of the specimens to prevent cross-sectional distortion from the large clamping force required to keep the specimen in place. Burrs were carefully removed using a metal file following the machining of both ends the specimens. The ferritic stub columns, tested at UPC, were laser cut before delivery by the supplier. These specimens were not deemed to require additional machining, due to the accurate cutting method and the use of a spherical head in the testing setup which is discussed further in Section 3.4.5.

Two tensile coupons were prepared from each cross-section, with the coupons having the traditional dog-bone shape and cut on opposite sides at 90° to the weld position. The tensile coupons were machined from 500 mm lengths of cut tube, this required 35 passes of a computer controlled cutting head removing a thin layer of material with each pass. To prevent the tube from vibrating and work hardening they were securely



Figure 3.6: A short CHS specimen being machined, with the aluminium insert visible.

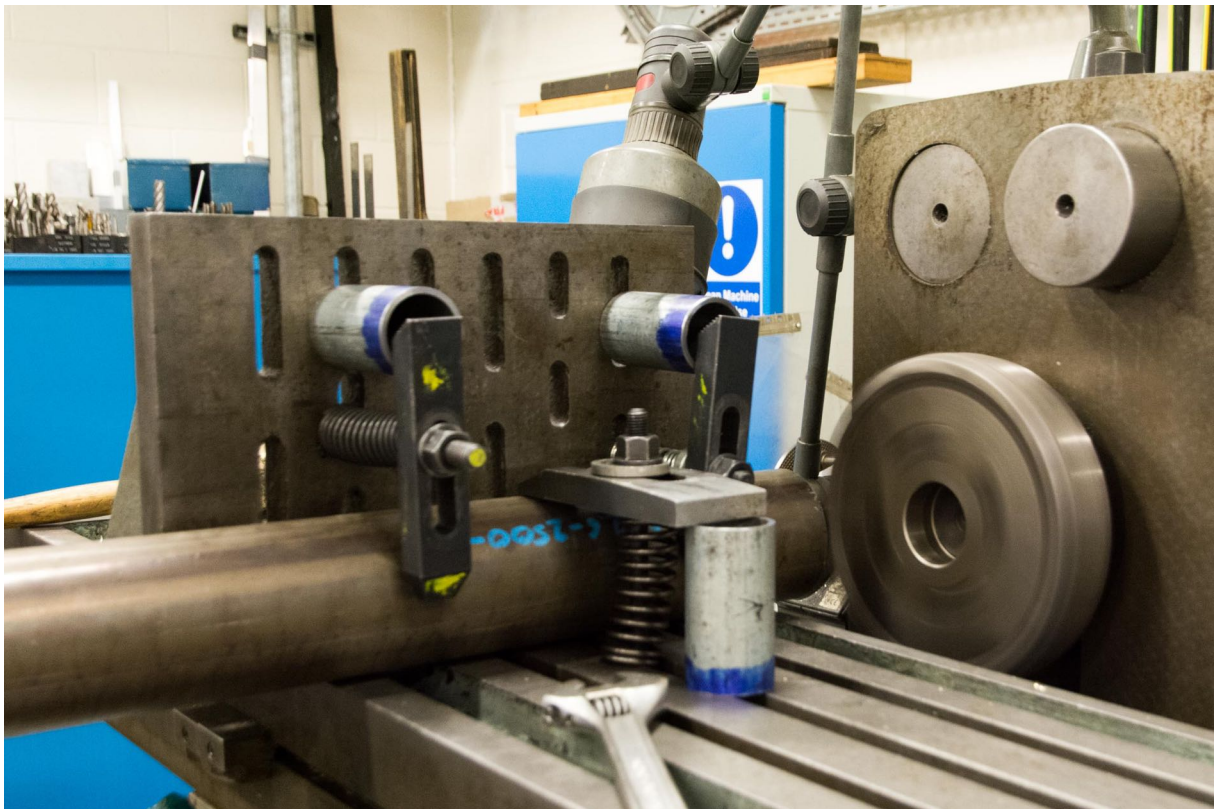


Figure 3.7: A long CHS specimen being machined.



Figure 3.8: A CHS coupon being machined, with the wooden block visible.

clamped and had a circular wooden block inserted inside the tube, as visible in Figure 3.8. The laboratory technicians undertaking the coupon machining noted that cutting head required frequent replacement due to the work hardening properties of stainless steel, and in particular for the ferritic grade tested.

3.4.3 Tensile coupon testing

The tensile coupon tests were undertaken in compliance with EN ISO 6892-1 (2009). The coupons were carefully measured using a digital calliper and had the standard gauge lengths marked using

$$L_o = k\sqrt{S_o} \not\leq 15 \text{ mm}, \quad (3.4)$$

from EN ISO 6892-1 (2009), where L_o is the original gauge length, k is the coefficient of proportionality, and took the typically adopted $k = 5.65$, and S_o is the cross-sectional

Table 3.3: CHS tensile coupon crosshead separation rates.

Strain rate $\dot{\epsilon}_{L_c}$ (/s)	Crosshead separation rate v_c (mm/s)	Coupon extension range (mm)
0.00007	0.01050	0 - 2.0
-	0.01725	2.0 - 2.1
-	0.02400	2.1 - 2.2
-	0.03075	2.2 - 2.3
0.00025	0.03750	2.3 -

area of the necked region.

An Instron 8802 testing machine was used, with data being recorded at a frequency of 1 Hz using a proprietary in-house developed datalogger and software. The instrumentation consisted of two mid-height electrical resistance strain gauges to measure strains until material yielding, a video extensometer that measured the strains after material yielding by monitoring the displacement of two painted white dots and a load cell to measure the applied tensile load. Black electrical tape was applied to the coupons to prevent reflections from interfering with the video extensometer measurements. A tensile coupon in the testing machine, just prior to the start of a test, is shown in Figure 3.9.

The testing rate was based on strain rate control (method A) from EN ISO 6892-1 (2009). As recommended two crosshead separation rates v_c were used before and post material yielding using

$$v_c = L_c \dot{\epsilon}_{L_c}, \quad (3.5)$$

where L_c is the parallel length of the coupon and $\dot{\epsilon}_{L_c}$ is the estimated strain rate over the parallel length. From EN ISO 6892-1 (2009) up to material yielding $\dot{\epsilon}_{L_c} = 0.00007s^{-1}$ and beyond yielding $\dot{\epsilon}_{L_c} = 0.00025s^{-1}$, with a gradual transition between the two rates as shown in Table 3.3. The parallel length used was the average measured parallel length, to standardise the displacement rate among the coupons.

‘Chemical metal’ filling material was applied to the concave face of the coupons to

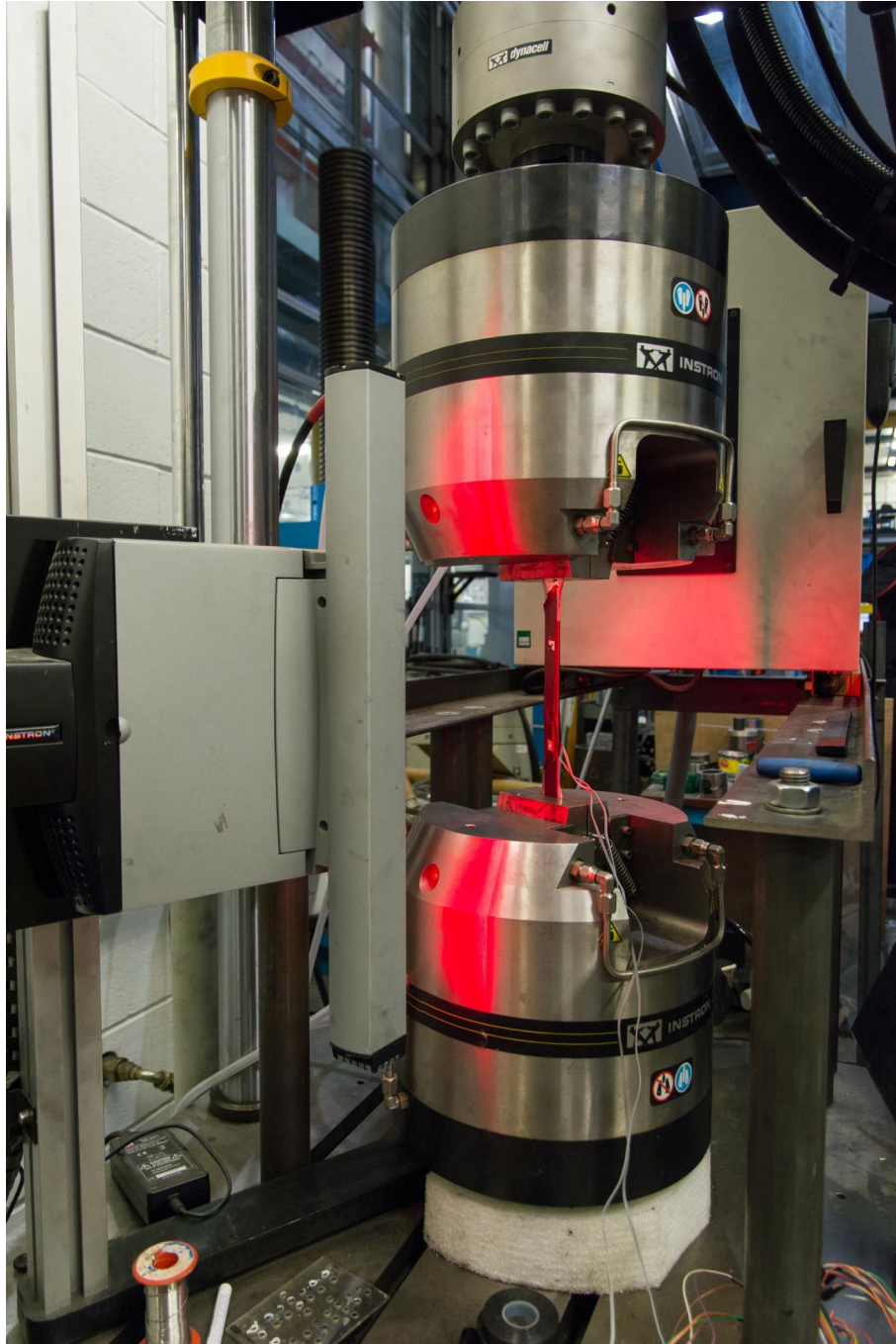


Figure 3.9: The 80×1.5-1 CHS tensile coupon in the testing machine, with strain measuring dots visible and black electrical tape reducing reflections.

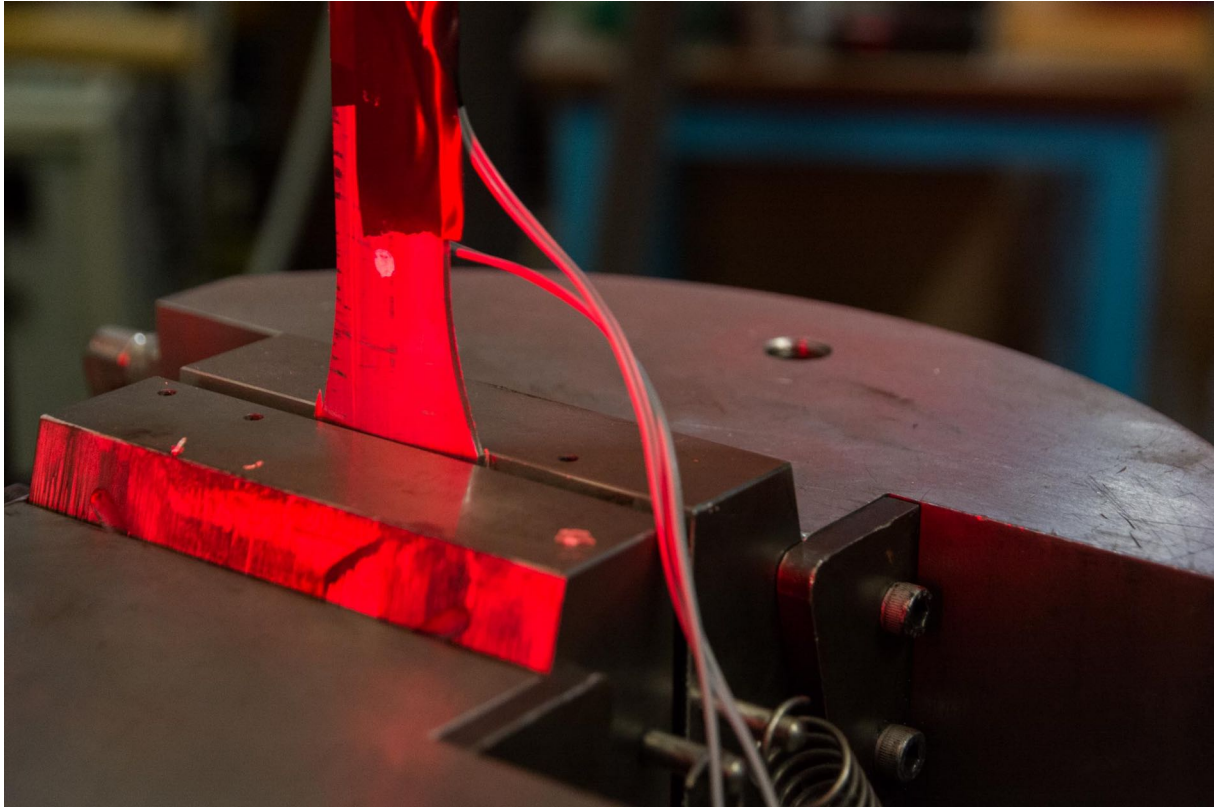


Figure 3.10: The CHS end radius maintained within the testing machine grip.

prevent the machine grips from deforming the ends of the coupons and the eccentrically applied tensile force inducing bending within the coupon. The curved profile of a tensile coupon, maintained within the gripping jaws, is apparent in Figure 3.10. The effect of the filling material on the shape of the coupon ends is also apparent in Figure 3.11, which shows i) a trial coupon without filling material being squashed flat by the gripping jaws, ii) a coupon with filling material after a test still with its original curved profile and iii) an untested coupon with the filling material applied. The CHS coupons were all curved longitudinally before testing (visible in Figure 3.11), which is the result of large through-thickness bending residual stresses (Rasmussen and Hancock, 1990).

The cross-sectional area of the necked region was determined using AutoCAD (Autodesk, 2015a), with the average measured coupon dimensions and the averaged outer diameter D and wall thickness t . Previous studies have measured the mass of a known length and then used the material density to determine the cross-sectional area (Afshan et al., 2013) and while this was attempted in this study, determining the cross-sectional area using

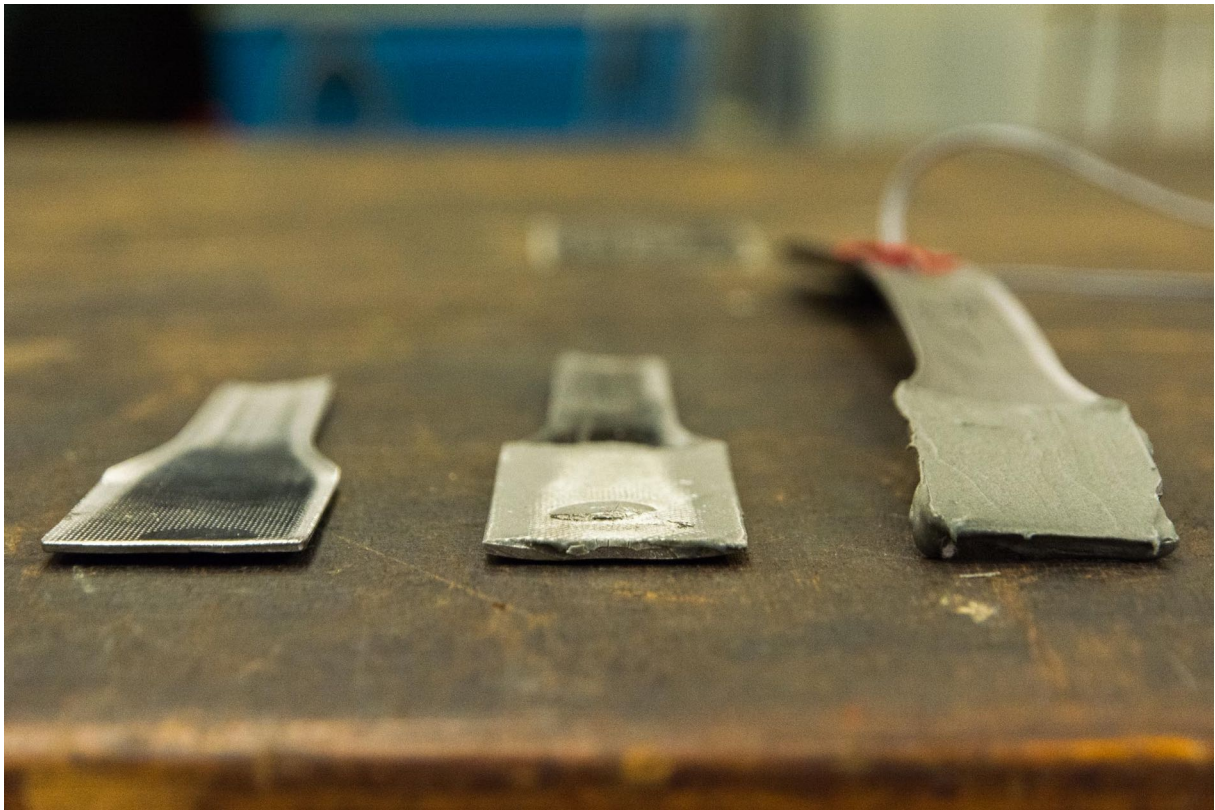


Figure 3.11: Left to right, i) a CHS coupon end without filling material after a test, ii) a CHS coupon with filling material after test with the initial radius maintained and iii) a CHS coupon with filling material prior to testing.

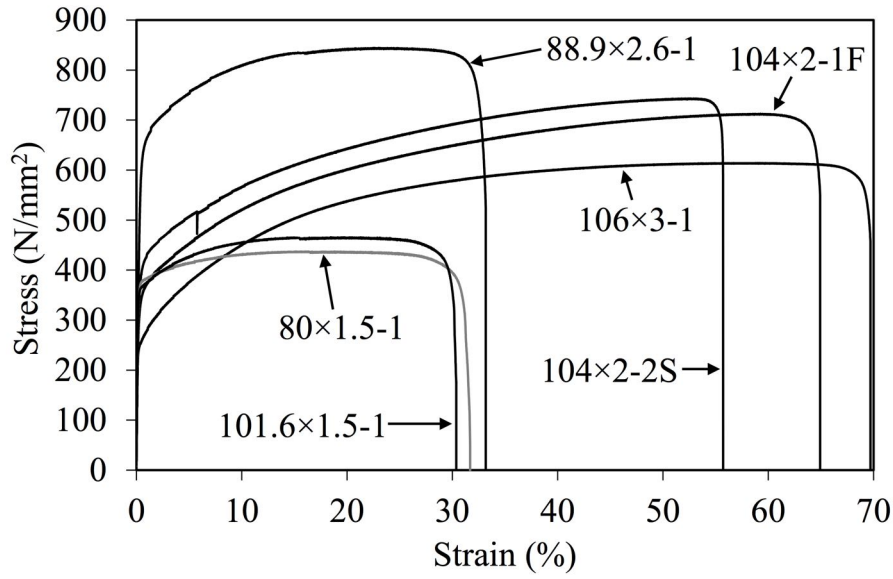


Figure 3.12: Typical tensile coupon stress–strain curves.

AutoCAD was found to be more accurate.

Typical stress–strain curves from the tensile coupons are shown in Figure 3.12 and the Young’s modulus E , 0.2% proof stress $\sigma_{0.2}$, 1.0% proof stress $\sigma_{1.0}$, ultimate tensile stress σ_u , strain at the ultimate tensile stress ε_u , fracture strain over the marked gauge length ε_f , the Ramberg–Osgood parameter n (Ramberg and Osgood, 1943) and the extended parameters $n'_{0.2,1.0}$ and $n'_{0.2,u}$ (Mirambell and Real, 2000; Rasmussen, 2003; Gardner and Ashraf, 2006) determined from the coupon tests are reported in Table 3.4. The 104×2 CHS tubes came from Sweden and Finland and additional tensile coupons were cut, labelled with ‘S’ and ‘F’ respectively. It is apparent from Table 3.4 that the ‘S’ coupons have a higher Young’s modulus E , 0.2% proof stress $\sigma_{0.2}$ and ultimate tensile stress σ_u than the ‘F’ coupons.

As discussed in the literature review, in Section 2.2.4, there is no precise yield point for stainless steel and hence the offset 0.2% proof stress $\sigma_{0.2}$ is used in place of the carbon steel yield stress f_y for structural design. Throughout this thesis the 0.2% proof stress $\sigma_{0.2}$ and yield stress f_y are considered to be interchangeable within equations.

Table 3.4: Summary of the tensile coupon material properties.

Coupon	Type	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ε_u (%)	ε_f (%)	n	$n'_{0.2,1.0}$	$n'_{0.2,u}$
106×3-1	A	226600	250	286	614	59.0	82.0	15.0	4.0	2.0
106×3-2	A	193600	272	309	616	60.1	84.8	8.0	3.3	1.9
104×2-1S	A	209600	346	407	733	54.9	76.8	6.7	2.9	2.4
104×2-2S	A	210700	378	437	743	52.6	75.2	6.4	2.7	2.4
104×2-1F	A	190800	331	379	712	59.3	73.6	7.3	2.9	2.1
104×2-2F	A	188900	328	378	717	61.7	72.6	6.1	3.0	2.1
88.9×2.6-1	D	196200	612	678	844	22.6	39.6	7.4	2.5	2.5
88.9×2.6-2	D	198200	637	684	849	23.4	37.5	10.3	2.4	2.2
80×1.5-1	F	229400	369	385	437	19.4	41.5	8.4	1.6	1.8
80×1.5-2	F	214700	373	388	439	13.7	47.1	9.0	1.6	1.7
101.6×1.5-1	F	227500	361	378	466	18.5	43.1	15.4	2.4	1.7
101.6×1.5-2	F	221100	355	373	468	19.4	45.5	13.5	2.4	1.7

3.4.4 Geometric properties

The geometric properties of the stub columns were carefully measured and recorded before testing. Outer diameter measurements were taken at three equally spaced longitudinal locations for the stub columns. At each location the outer diameter was recorded in four evenly distributed orientations (at 45° intervals) with callipers, allowing the average outer diameter D of the specimen to be calculated along with its ‘out-of-roundness’ as defined in EN 10219-2 (2006a), and reproduced as Equation 3.3 in Section 3.4.2. The average wall thickness t was determined from measurements of the wall thickness at the ends of the tubes at eight equally spaced locations (at 45° intervals) around the circumference, at a distance not less than $2t$ from the weld location in accordance with EN 10219-2 (2006a). The average specimen length L was calculated by averaging the specimen length from four separate measurements taken at 90° locations around the circumference. The stub column lengths were chosen to be sufficiently long to include representative local imperfections and residual stresses, but short enough to preclude global buckling (Ziemian, 2010). The mid-point global imperfections were recorded by measuring the deviation from a flat plane linking the two ends of the specimens. The deviation was measured using the bottom of a spirit level and feeler gauges. The average geometric properties of the stub columns are provided in Table 3.5. The specimen notation has been chosen for clarity, for example 104×2-400-F is the 104×2 CHS cross-section with 400 mm nominal length for the stub columns and ‘F’ fixed end conditions, and a specimen ending with ‘R’ denotes a repeat specimen.

Square grids were painted onto the specimens prior to testing to make it easier to observe the deformations arising from the stub column tests. Specimen labels were moved to the inside of the tubes, the outer surface was thoroughly cleaned using acetone and then a grid was painted using the same lathe machine used to grind the ends flat and parallel. A paint pen could be clamped in the cutting head position and could be moved horizontally, which in conjunction with hand rotation of the specimen allowed a square grid to be drawn on the surface, as shown in Figure 3.13.

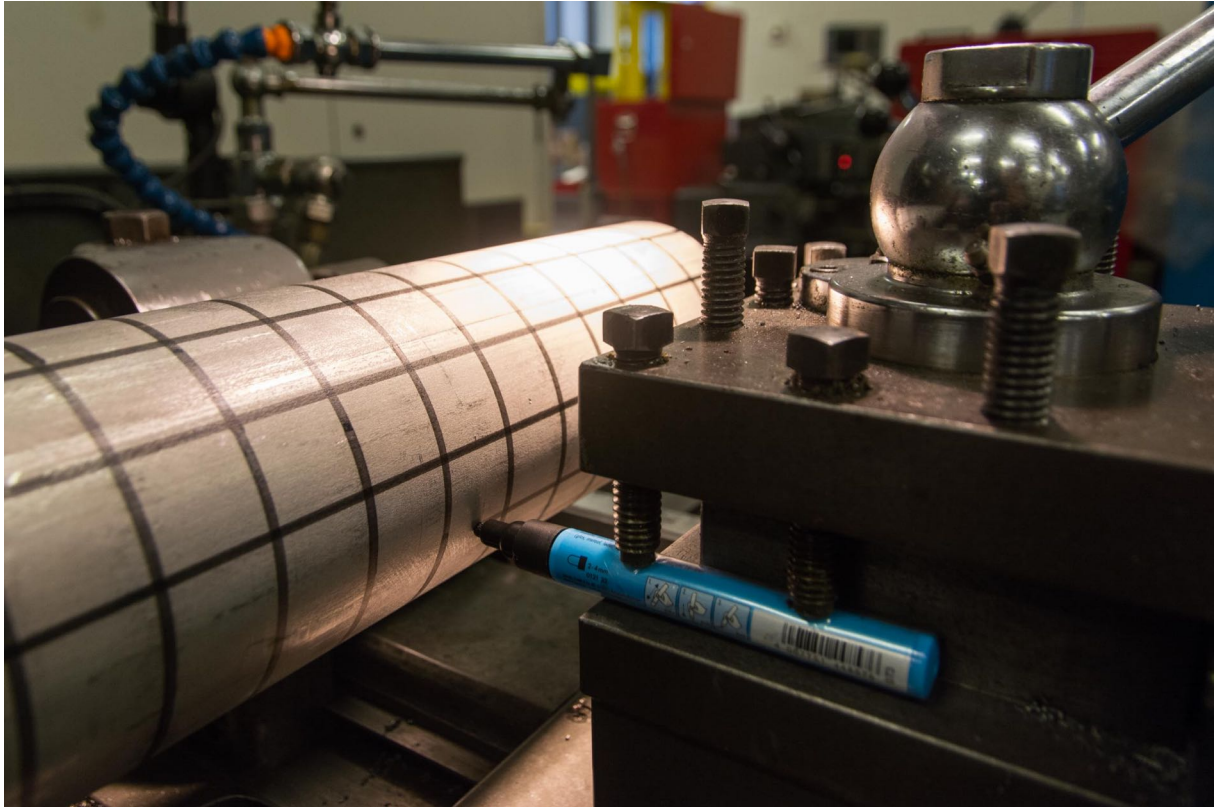


Figure 3.13: The CHS grid painting setup.

3.4.5 Stub column test setup

The test setup at ICL for the austenitic and duplex stub columns is shown in Figure 3.14. An Instron 3500kN-SPL testing machine was used with a proprietary in-house developed datalogger and software recording the various parameters at a recording frequency of 2 Hz. The instrumentation consisted of three electrical resistance strain gauges equally spaced around the circumference at mid-height (with the weld located centrally between two strain gauges), three equally spaced potentiometers to measure the vertical movement of the top platen and a load cell within the machine to measure the applied load.

The testing setup for the ferritic stub columns at UPC is shown in Figure 3.15. An Instron 8805 testing machine was used with a HBM MGCplus datalogger and Catman software

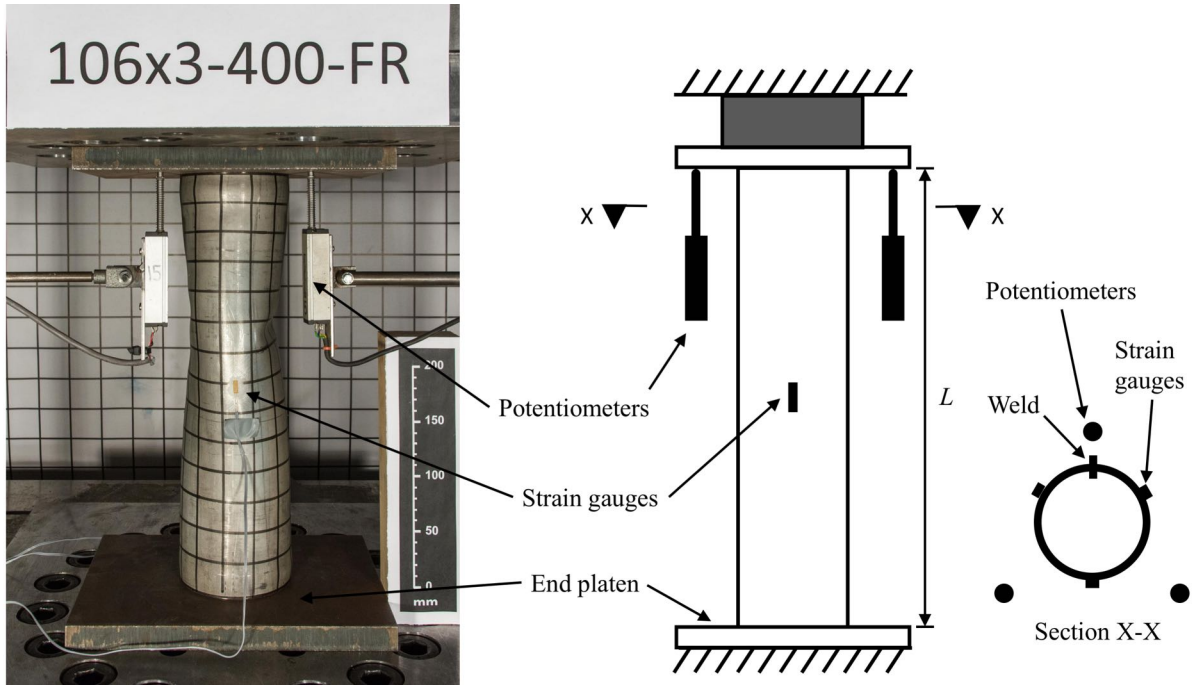


Figure 3.14: The ICL stub column test setup.

recording at a frequency of 2 Hz. A spherical head was utilised to ensure full contact with the top of the stub columns, with the head free to move initially but locked at a load of 5 kN, ensuring full contact of both ends of the stub column with the testing machine. The instrumentation consisted of three electrical resistance strain gauges equally spaced around the circumference at mid-height (with the weld located centrally between two strain gauges), three equally spaced linear variable differential transformer (LVDTs) to measure the movement of the top platen, and a load cell within the testing machine to measure the applied compressive load. The strain gauges were all attached after the strain visualisation grids had been painted, with their locations having been prepared prior to the grid being applied.

The top platen moved at a displacement rate of 0.2 mm/min for the austenitic and ferritic stub columns, and at a slightly slower displacement rate of 0.15 mm/min for the shorter duplex 88.9×2.6 CHS stub columns, in line with previous CHS stub column tests by Stangenberg (2000) and Zhao et al. (2015a) and were continued beyond the ultimate load to observe the initial unloading behaviour. Two stub columns were tested for each cross-section, with one from each country of origin for the 104×2 CHS stub columns.

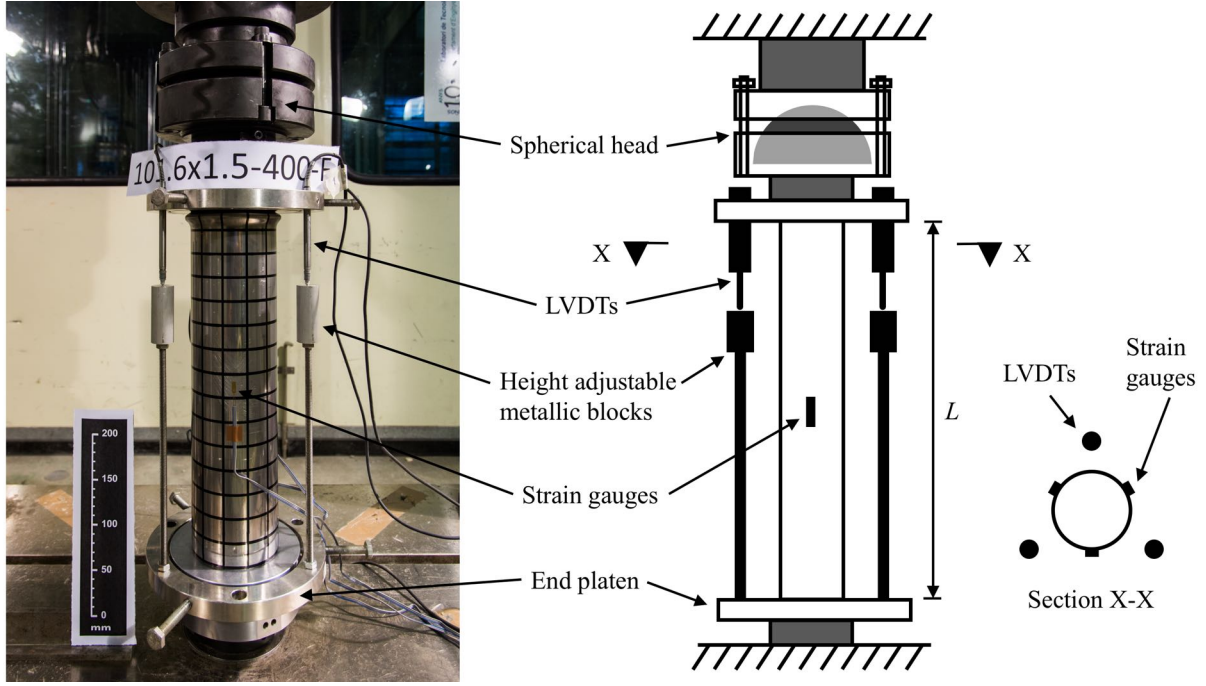


Figure 3.15: The UPC stub column test setup.

3.4.6 Stub column test results

The average outer diameter D , wall thickness t , specimen length L , initial imperfection ω_0 , ultimate axial load N_u , end shortening at the ultimate load δ_u , accounting for the deformation of the end platens (Centre for Advanced Structural Engineering, 1990), ultimate load normalised by the yield load $N_u/(A\sigma_{0.2})$ (where $\sigma_{0.2}$ is the average tensile 0.2% proof stress) and the global slenderness $\bar{\lambda}$ (determined using tensile properties) are reported in Table 3.5. The global slenderness $\bar{\lambda}$ is determined using

$$\bar{\lambda} = \sqrt{\frac{A\sigma_{0.2}L^2}{\pi^2 EI}} \text{ for class 1, 2 and 3 cross-sections} \quad (3.6)$$

and

$$\bar{\lambda} = \sqrt{\frac{A_{\text{eff}}\sigma_{0.2}L^2}{\pi^2 EI}} \text{ for class 4 cross-sections,} \quad (3.7)$$

where A is the gross cross-sectional area, A_{eff} is the effective cross-sectional area, $\sigma_{0.2}$ is

the 0.2% proof stress, E is the Young's modulus, I is the second moment of area and L is the effective length. EN 1993-1-4 (2015) does not provide an expression for A_{eff} for a CHS and therefore the design formula from BS 5950-1 (2000) has been used with updated EN 1993-1-4 (2015) compressive class 3 slenderness limits (Chan and Gardner, 2008b; Buchanan et al., 2016a),

$$A_{\text{eff}} = A \left[\left(\frac{90}{D/t} \right) \left(\frac{235}{f_y} \frac{E}{210000} \right) \right]^{0.5}. \quad (3.8)$$

The true end shortening e_{True} can be determined using

$$e_{\text{True}} = e_{\text{LVDT}} - 2\delta_{\text{Platen}}, \quad (3.9)$$

where e_{LVDT} is the average end shortening measured by the LVDT and δ_{Platen} is the end platen deformation from

$$\delta_{\text{Platen}} = \frac{L}{2} \sigma \left(\frac{1}{E_{0,\text{LVDT}}} - \frac{1}{E_{0,\text{True}}} \right), \quad (3.10)$$

where L is the stub column length, σ is the applied stress and $E_{0,\text{LVDT}}$ and $E_{0,\text{True}}$ are the Young's moduli determined using the LVDT and strain gauge readings respectively.

The country of origin for the 104×2 CHS is also listed, with FIN and SWE being Finland and Sweden respectively. The compressive material properties derived from the stub column tests, which were deemed not to be influenced by local buckling since the peak strains were well in excess of those at the 0.2% proof stress, (E_{stub} , $\sigma_{0.2,\text{stub}}$ and n_{stub}) are listed in Table 3.6. The average tensile 0.2% proof stress normalised by the compressive stub column 0.2% proof stress $\sigma_{0.2}/\sigma_{0.2,\text{stub}}$ shows that the tensile and compressive properties are generally consistent but, on average, the compressive 0.2% proof stress is 1% below that in tension. The stub column load–end shortening curves are shown in Figure 3.16. The load–end shortening curves show that the class 1 106×3 CHS cross-sections undergo significant strain hardening before local buckling. The 104×2, 88.9×2.6, 80×1.5 and 101.6×1.5 CHS stub columns all exhibited the classic

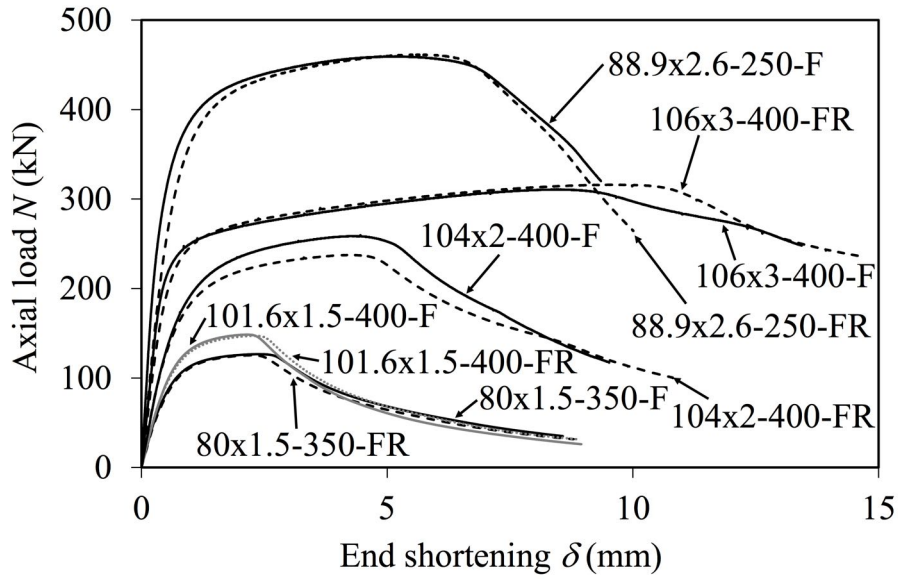


Figure 3.16: Stub column load–end shortening curves.

‘elephant’s foot’ local buckle at one end (visible in Figure 3.15), while the 106×3 CHS stub columns initially developed a slight ‘elephant’s foot’ buckle but locally buckled inwards close to mid-height as shown in Figure 3.14. The deformed stub columns are shown in Figures 3.17 and 3.18.

The number of experimental results used in the definition and assessment of the various aspects of the CSM for CHS, i) the yield slenderness limit (see Section 3.5); ii) the base curve for non-slender sections (see Section 3.7); iii) the base curve for slender sections (see Section 3.7); and iv) the assessment of the capacity predictions (see Section 3.10) are shown in Tables 3.7 and 3.8 for compression and bending respectively. The number of specimens used in the different stages of the extension of the CSM to CHS sometimes varies since not all required parameters were reported in the literature.

3.5 Yield slenderness limit

The limiting local slenderness that delineates the transition between slender and non-slender cross-sections needs to be defined. Above this limit there is no significant benefit

Table 3.5: Summary of the measured geometric properties and test results for the stub columns.

Specimen	Type	Origin	D (mm)	t (mm)	L (mm)	ω_0 (mm)	N_u (kN)	δ_u (mm)	$N_u/(A\sigma_{0.2})$	Global slenderness $\bar{\lambda}$
106×3-400-F	A	-	105.76	2.88	400.14	0.15	310.9	8.34	1.23	0.06
106×3-400-FR	A	-	105.70	2.79	400.24	0.40	316.0	9.59	1.34	0.06
104×2-400-F	A	FIN	104.02	1.85	399.87	0.15	259.6	4.46	1.33	0.07
104×2-400-FR	A	SWE	104.09	1.86	399.76	- ^a	237.5	4.57	1.10	0.07
88.9×2.6-250-F	D	-	88.65	2.35	249.93	0.30	459.6	5.02	1.04	0.07
88.9×2.6-250-FR	D	-	88.69	2.42	249.88	0.20	461.4	5.55	1.05	0.07
80×1.5-350-F	F	-	80.00	1.34	350	0.05	126.7	2.41	1.03	0.08
80×1.5-350-FR	F	-	80.00	1.34	350	0.15	125.4	2.25	1.02	0.08
101.6×1.5-400-F	F	-	101.75	1.33	400	0.40	148.5	2.18	0.99	0.07
101.6×1.5-400-FR	F	-	101.80	1.34	400	0.25	147.2	2.31	0.97	0.07

Table 3.6: Summary of the stub column material properties.

Specimen	Type	Origin	E_{stub} (N/mm ²)	$\sigma_{0.2,\text{stub}}$ (N/mm ²)	η_{stub}	$\sigma_{0.2}/\sigma_{0.2,\text{stub}}$	EN 1993-1-4 (2015) compressive class
106×3-400-F	A	-	188700	276	9.4	0.95	1
106×3-400-FR	A	-	203400	291	7.8	0.90	1
104×2-400-F	A	FIN	199700	373	4.6	0.88	3
104×2-400-FR	A	SWE	204400	346	5.0	1.05	3
88.9×2.6-250-F	D	-	220200	590	4.6	1.06	4
88.9×2.6-250-FR	D	-	215700	569	4.4	1.10	4
80×1.5-350-F	F	-	219600	361	5.1	1.03	3
80×1.5-350-FR	F	-	217900	359	5.1	1.03	3
101.6×1.5-400-F	F	-	218400	341	5.7	1.05	4
101.6×1.5-400-FR	F	-	220700	333	5.5	1.07	4

^a Initial imperfection less than 0.05 mm



Figure 3.17: The deformed 106×3-400-F, 106×3-400-FR, 104×2-400-F, 104×2-400-FR, 88.9×2.6-250-F and 88.9×2.6-250-FR CHS stub columns, from left to right.

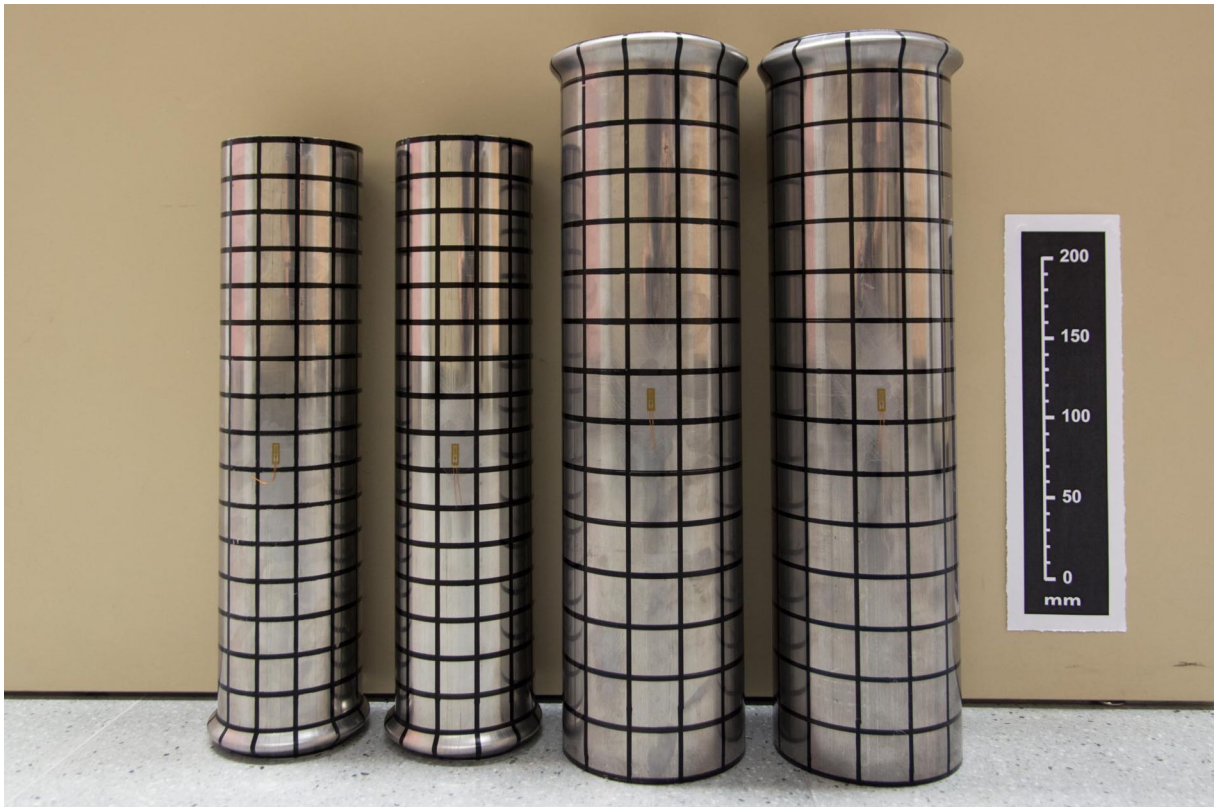


Figure 3.18: The deformed 80×1.5-350-F, 80×1.5-350-FR, 101.6×1.5-400-F and 101.6×1.5-400-FR CHS stub columns, from left to right.

Table 3.7: The number of CHS stub column test results used in the development of the various aspects of the CSM.

Material	Yield slenderness limit	Base curve		CSM and code capacity predictions	
		Non-slender sections	Slender sections	Non-slender sections	Slender sections
Hot-finished structural steel	11	8	-	9	-
Very high strength structural steel	20	1	14	1	19
Cold-formed structural steel	131	48	50	44	52
Stainless steel (total)	86	37	30	45	39
Austenitic stainless steel	52	20	13	28	22
Duplex stainless steel	23	10	13	10	13
Ferritic stainless steel	11	7	4	7	4
Aluminium	15	7	7	7	8

Table 3.8: The number of CHS four-point bending test results used in the development of the various aspects of the CSM.

Material	Yield slenderness limit	Base curve		CSM and code capacity predictions	
		Non-slender sections	Slender sections	Non-slender sections	Slender sections
Hot-finished structural steel	14	2	-	14	-
Very high strength structural steel	12	1	8	1	11
Cold-formed structural steel	44	9	17	13	27
Stainless steel (total)	12	1	8	3	9
Austenitic stainless steel	8	1	4	3	5
Duplex stainless steel	4	-	4	-	4
Ferritic stainless steel	-	-	-	-	-
Aluminium	7	3	4	3	4

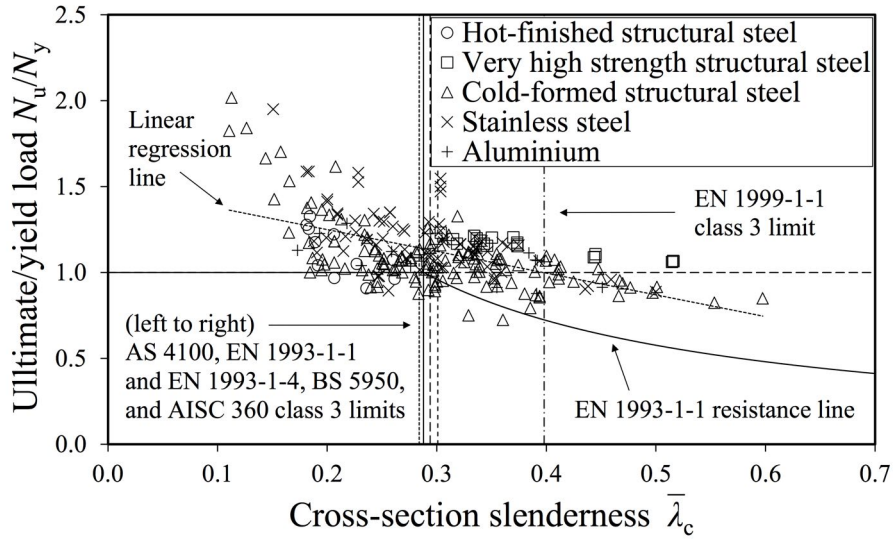


Figure 3.19: Normalised ultimate axial resistance N_u/N_y varying with local slenderness $\bar{\lambda}_c$.

from strain hardening with the cross-section buckling locally below the yield load in compression or elastic moment in bending. This limit is identified by plotting the ultimate capacity of the stub columns normalised by their yield load (N_u/N_y) against cross-section slenderness $\bar{\lambda}_c$, defined by Equation 3.1, as shown in Figure 3.19. A linear regression fit can then identify the limiting local slenderness where the ultimate axial load equals the yield load, which from Figure 3.19 is $\bar{\lambda}_c \approx 0.40$. The class 3 limits from current design codes are also plotted in Figure 3.19, and it can be seen that the identified limiting local slenderness is compatible with the class 3 limit for aluminium given in EN 1999-1-1 (2007); however it is above the existing structural carbon steel and stainless steel class 3 limits. There is also some scatter in the stub column dataset. Consequently, a lower value of $\bar{\lambda}_c = 0.3$ for the yield slenderness limit is proposed as this represents approximately a lower bound to the assembled dataset and is generally comparable with existing codes.

It is evident from Figure 3.19 that there are no clear discontinuities in the dataset and that limiting the maximum material stress to the yield stress is overly conservative for stocky cross-sections. The ultimate bending moments from the collected beam tests normalised by the elastic moment (M_u/M_{el}) are plotted in Figure 3.20 against the cross-section

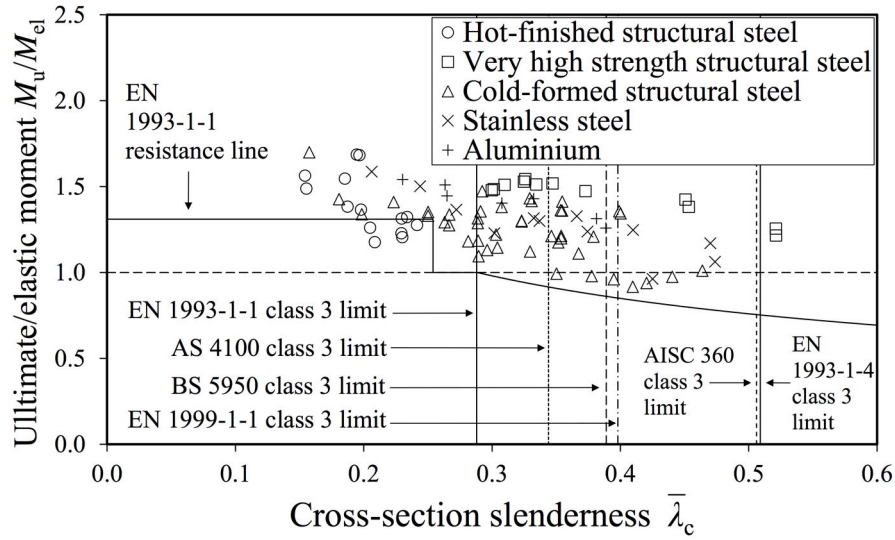


Figure 3.20: Normalised ultimate flexural resistance M_u/M_{el} varying with local slenderness $\bar{\lambda}_c$.

slenderness. As for compression, in contrast to the EN 1993-1-1 (2014) resistance line, there are no apparent discontinuities in the bending capacity dataset that cross-section classification would otherwise suggest and again limiting the material stress to the yield stress can lead to under-predictions of the ultimate cross-section capacity for stocky sections. The results show that the previous yield slenderness limit of $\bar{\lambda}_c = 0.3$ can also be applied for bending.

3.6 Normalised deformation capacity (strain ratio)

In the CSM, cross-section classification is replaced by a continuous relationship between local slenderness and deformation capacity. This deformation capacity is called the strain ratio ($\varepsilon_{csm}/\varepsilon_y$) and is defined as the strain at ultimate load ε_{csm} , which is taken as the limiting strain for the cross-section, normalised by the yield strain ε_y . The strain ratio is determined from stub column and four-point bending experiments, as described below.

3.6.1 Axial compression

For stub columns with a local slenderness below the yield slenderness limit ($\bar{\lambda}_c \leq 0.3$) and where the ultimate load exceeds the yield load ($N_u \geq N_y$), the strain ratio is expressed as a function of the strain at ultimate load divided by the yield strain ($\varepsilon_{lb}/\varepsilon_y$),

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{\varepsilon_{lb} - 0.002}{\varepsilon_y} = \frac{\delta_u/L - 0.002}{\varepsilon_y} \quad (3.11)$$

for materials with a rounded stress–strain material response, with a 0.002 strain offset is subtracted from ε_{lb} for compatibility with the material models adopted in Section 3.8, and

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{\varepsilon_{lb}}{\varepsilon_y} = \frac{\delta_u/L}{\varepsilon_y} \quad (3.12)$$

for materials with a sharply defined yield point. The strain at ultimate load ε_{lb} is calculated from the initial specimen length L and the end–shortening δ_u at the ultimate load N_u .

For specimens that do not exceed the yield load ($N_u < N_y$) or that have slender cross-sections ($\bar{\lambda}_c > 0.3$), the strain ratio is determined as the ratio of the ultimate load to yield load (N_u/N_y),

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{N_u}{N_y}. \quad (3.13)$$

3.6.2 Four-point bending

Under uniform bending, the strain ε can be determined as the product of the curvature κ and the distance from the elastic neutral axis y ,

$$\varepsilon = \kappa y. \quad (3.14)$$

The curvature at the ultimate moment and the elastic moment are termed κ_u and κ_{el}

respectively. κ_u can be calculated from existing four-point bending experimental data using Equation 3.15,

$$\kappa = \frac{1}{r} = \frac{8(D_M - D_L)}{4(D_M - D_L)^2 + L_2^2} \quad (3.15)$$

where r is the radius of curvature, D_M and D_L are the displacements at the midspan and loading points respectively and L_2 is the length of the central region of the beam between the two loading points (Chan and Gardner, 2008a). The curvature at the elastic moment κ_{el} can be determined from Equation 3.16,

$$\kappa_{el} = \frac{M_{el}}{EI} \quad (3.16)$$

where I is the second moment of area.

The strain ratio for four-point bending is similar in principle to axial compression, being defined as a function of the maximum strain in the cross-section at the ultimate moment normalised by the yield strain ($\varepsilon_{lb}/\varepsilon_y$) for $\bar{\lambda}_c \leq 0.3$ and where the ultimate moment exceeds the elastic moment ($M_u \geq M_{el}$),

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{\varepsilon_{lb} - 0.002}{\varepsilon_y} = \frac{\kappa_u y_{max} - 0.002}{\kappa_{el} y_{max}} \quad (3.17)$$

with a 0.002 offset again subtracted for materials with a rounded stress-strain response and

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{\varepsilon_{lb}}{\varepsilon_y} = \frac{\kappa_u y_{max}}{\kappa_{el} y_{max}} \quad (3.18)$$

for materials with a sharply defined yield point, where y_{max} is the distance from the elastic neutral axis to the extreme fibre of the cross-section.

If the cross-section is slender ($\bar{\lambda}_c > 0.3$) or the ultimate moment is less than the elastic moment ($M_u < M_{el}$), the strain ratio is taken as the ultimate moment normalised by the

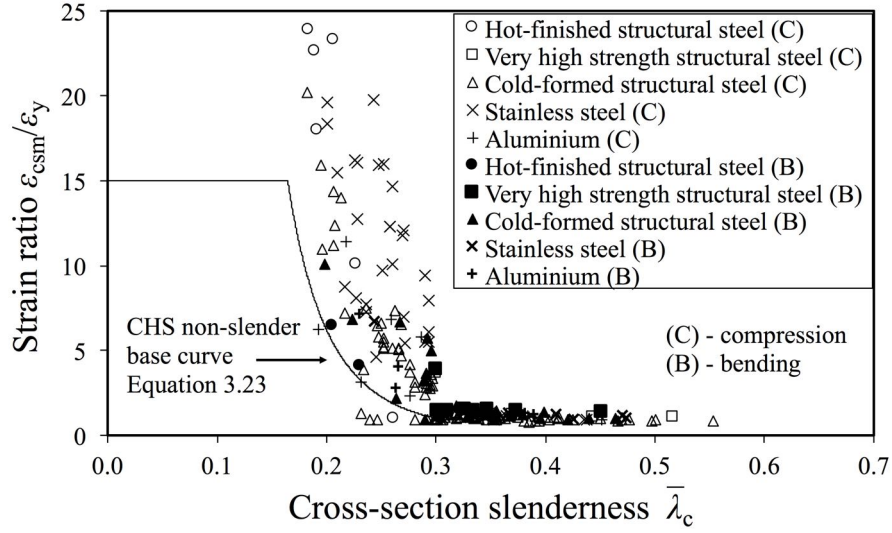


Figure 3.21: The CHS CSM non-slender base curve with the collected experimental data.

elastic moment (M_u/M_{el}),

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{M_u}{M_{el}}. \quad (3.19)$$

3.7 Base curve

The base curve defines the relationship between the deformation capacity of a cross-section (i.e. the maximum limiting strain that it can endure ε_{csm}) and the local slenderness of the cross-section. Base curves for non-slender ($\bar{\lambda}_c \leq 0.3$) CHS,

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{A}{\bar{\lambda}_c^B} \text{ for } \bar{\lambda}_c \leq 0.3, \quad (3.20)$$

and slender ($\bar{\lambda}_c > 0.3$) CHS,

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{F}{\bar{\lambda}_c^G}\right) \frac{1}{\bar{\lambda}_c^G} \text{ for } \bar{\lambda}_c > 0.3, \quad (3.21)$$

can be fitted to the experimental strain ratios derived from the axial and bending results from the literature and new tests (Tables 3.7 and 3.8), as plotted in Figures 3.21 and 3.22. Equation 3.20 is consistent with previous implementations of the CSM, and is similar in form to the relationship between normalised elastic buckling strain $\varepsilon_{cr}/\varepsilon_y$ and local

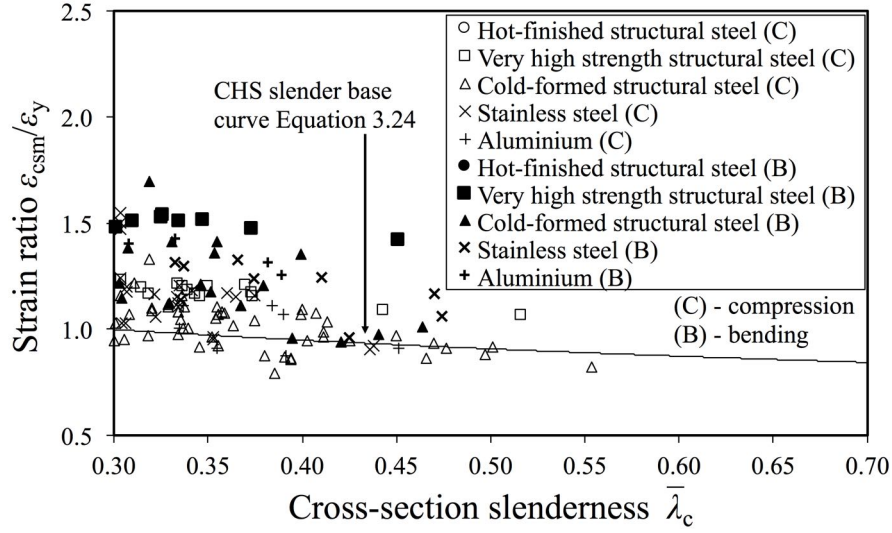


Figure 3.22: The CHS CSM slender base curve with the collected experimental data.

slenderness $\bar{\lambda}_c$, given by

$$\frac{\varepsilon_{cr}}{\varepsilon_y} = \frac{1}{\bar{\lambda}_c^2}. \quad (3.22)$$

Equation 3.21 is of the same general form as the normalised strength curves from the Direct Strength Method (DSM) (American Iron and Steel Institute, 2004). The two chosen base curves for non-slender CHS,

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{4.44 \times 10^{-3}}{\bar{\lambda}_c^{4.5}} \text{ for } \bar{\lambda}_c \leq 0.3 \text{ but } \frac{\varepsilon_{csm}}{\varepsilon_y} \leq \min \left(15, \frac{C_1 \varepsilon_u}{\varepsilon_y} \right), \quad (3.23)$$

and for slender CHS,

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{0.224}{\bar{\lambda}_c^{0.342}} \right) \frac{1}{\bar{\lambda}_c^{0.342}} \text{ for } 0.3 < \bar{\lambda}_c \leq 0.6, \quad (3.24)$$

generally represent a lower bound to the dataset and meet at the yield slenderness limit ($\bar{\lambda}_c = 0.3$) previously identified at a strain ratio $\varepsilon_{csm}/\varepsilon_y$ of unity.

Upper limits are placed upon the strain ratio, in effect limiting the extent to which the non-slender cross-sections can deform. The upper limit of 15 in Equation 3.23 is the material ductility requirement from EN 1993-1-1 (2014) and is applied to all

metallic materials. A second upper limit is applied to cold-formed and very high strength structural carbon steel ($C_1 = 0.4$), austenitic and duplex stainless steel ($C_1 = 0.1$), ferritic stainless steel ($C_1 = 0.4$) and aluminium ($C_1 = 0.5$) to prevent over-predictions of cross-section resistance due to the chosen simplified material model. ε_u is the strain at the ultimate tensile stress of the material, and is discussed further in Section 3.8. An upper limit of $\bar{\lambda}_c \leq 0.6$ is placed upon the slender base curve since experimental data has not been examined beyond this slenderness.

3.8 Material models

An elastic, linear strain hardening material model (of slope E_{sh}), as shown in Figure 3.23, is adopted in the CSM, replacing the traditional elastic, perfectly-plastic material model typically employed in existing design codes. The coefficients C_1 to C_4 are defined in Section 3.8.5. The CSM limiting stress f_{csm} is defined by

$$f_{csm} = E\varepsilon_{csm} \text{ for } \frac{\varepsilon_{csm}}{\varepsilon_y} < 1 \quad (3.25)$$

and by

$$f_{csm} = f_y + E_{sh}\varepsilon_y \left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1 \right) \text{ for } \frac{\varepsilon_{csm}}{\varepsilon_y} \geq 1, \quad (3.26)$$

which represents the strain hardening behaviour of the material through the strain hardening modulus E_{sh} .

3.8.1 Hot-finished structural carbon steel sections

For hot-finished structural carbon steel sections, the strain hardening modulus E_{sh} proposed by Foster (2014) has been utilised. This is the simplest model adopted as strain hardening is taken as zero,

$$\frac{E_{sh}}{E} = 0, \quad (3.27)$$

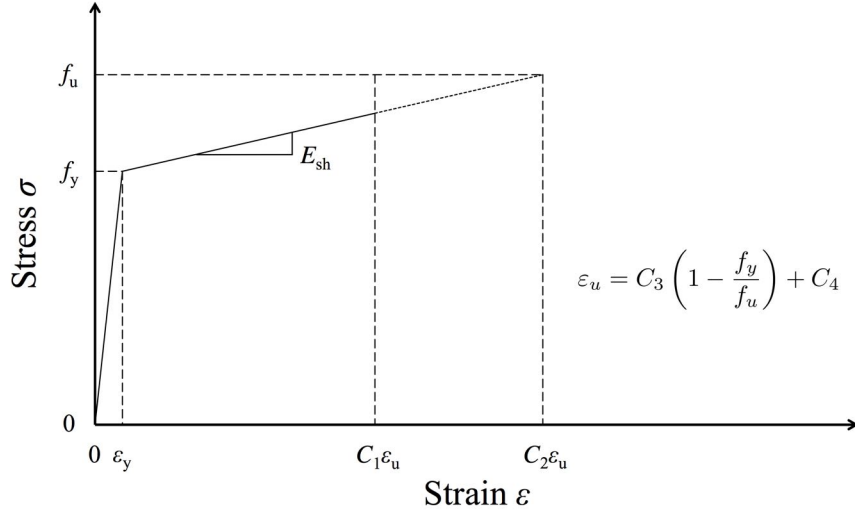


Figure 3.23: The elastic, linear strain hardening CSM material model.

reducing the stress–strain response to the traditional linear elastic, perfectly-plastic model. This is due to the extensive yield plateau associated with hot-finished tubes. Work is currently underway to develop a more refined material model for hot-finished structural carbon steel sections that takes into account strain hardening at the end of the yield plateau, leading to strain hardening benefits for cross-sections with high deformation capacities.

3.8.2 Cold-formed structural carbon steel sections

For cold-formed structural carbon steel sections, based on recent work by Gardner et al. (2015), the strain hardening modulus E_{sh} is defined by

$$E_{sh} = \frac{f_u - f_y}{0.45\varepsilon_u - \varepsilon_y} \text{ for } \varepsilon_y/\varepsilon_u < 0.45 \quad (3.28)$$

and

$$E_{sh} = 0 \text{ for } \varepsilon_y/\varepsilon_u \geq 0.45, \quad (3.29)$$

depending on the ratio $\varepsilon_y/\varepsilon_u$. This model is also applied to the very high strength structural carbon steel CHS considered herein. The material strain ε_u corresponding to

the ultimate tensile stress f_u may be predicted using

$$\varepsilon_u = 0.6 \left(1 - \frac{f_y}{f_u} \right). \quad (3.30)$$

3.8.3 Stainless steel sections

For stainless steel sections, the material model developed by Afshan and Gardner (2013b) is utilised for the austenitic and duplex grades. The strain hardening modulus E_{sh} is predicted from

$$E_{sh} = \frac{f_u - f_y}{0.16\varepsilon_u - \varepsilon_y} \quad (3.31)$$

and ε_u is given by

$$\varepsilon_u = 1 - \frac{f_y}{f_u}, \quad (3.32)$$

which is taken from EN 1993-1-4 (2015), based on the work by Rasmussen (2003). The ultimate tensile stress f_u , if not provided, can be estimated using

$$f_u = \frac{f_y}{0.2 + 185f_y/E}, \quad (3.33)$$

also from Rasmussen (2003).

The material model developed by Bock et al. (2015b) has been applied to the ferritic grades. This model is the same as that proposed for cold-formed structural carbon steel sections, and therefore utilises Equations 3.28, 3.29 and 3.30 (Bock et al., 2015b). If f_u is not provided (Real et al., 2014; Arrayago et al., 2015) it can be predicted using

$$f_u = \frac{f_y}{0.46 + 145f_y/E}. \quad (3.34)$$

3.8.4 Aluminium sections

For aluminium alloy sections, the material model proposed by Su et al. (2014b) is adopted in this study. The predictive expression for the strain hardening modulus E_{sh} is given by

$$E_{sh} = \frac{f_u - f_y}{0.5\varepsilon_u - \varepsilon_y}. \quad (3.35)$$

This is similar in form to the previous cold-formed structural carbon steel and stainless steel models. The material ultimate strain ε_u may be predicted from

$$\varepsilon_u = 0.13 \left(1 - \frac{f_y}{f_u} \right) + 0.06 \text{ for } f_u/f_y > 1.01. \quad (3.36)$$

3.8.5 Summary of material models

The CSM provides a harmonised design approach across all metallic materials. A common base curve (Equations 3.23 and 3.24) is used for CHS of all materials, while the differing strain hardening characteristics of the various materials are accounted for through different coefficients in the general elastic, linear hardening $\sigma - \varepsilon$ curve (see Figure 3.23). There are four coefficients C_1 , C_2 , C_3 and C_4 ; C_1 defines a ‘cut-off’ strain (in Equation 3.23) to avoid over-predictions of material strength when using the elastic, linear hardening material model; C_2 is used to define the strain hardening slope E_{sh} ,

$$E_{sh} = \frac{f_u - f_y}{C_2\varepsilon_u - \varepsilon_y}, \quad (3.37)$$

and C_3 and C_4 are used in the predictive expression for ultimate strain,

$$\varepsilon_u = C_3 \left(1 - \frac{f_y}{f_u} \right) + C_4, \quad (3.38)$$

which is needed to determine E_{sh} . Values of the coefficients for the different considered materials are summarised in Table 3.9.

Table 3.9: Summary of the coefficients for the CSM material model.

	C_1	C_2	C_3	C_4
Hot-finished structural steel	-1	-1	-1	-1
Very high strength structural steel	0.40	0.45	0.60	0
Cold-formed structural steel	0.40	0.45	0.60	0
Austenitic and duplex stainless steel	0.10	0.16	1.00	0
Ferritic stainless steel	0.40	0.45	0.60	0
Aluminium	0.50	0.50	0.13	0.06

¹ Hot-finished structural steel material model under development

3.9 CSM cross-section resistance functions for CHS

The cross-section resistances in compression or bending can now be determined utilising the deformation capacity ($\varepsilon_{\text{csm}}/\varepsilon_y$) predicted from the base curve, together with the adopted material model.

3.9.1 Compressive resistance

The CSM axial compressive resistance for non-slender cross-sections N_{csm} is calculated as the product of the gross cross-section area A and the CSM limiting material stress f_{csm} , as given by

$$N_{\text{csm}} = Af_{\text{csm}} \text{ for } \bar{\lambda}_c \leq 0.3. \quad (3.39)$$

The strength benefit for non-slender cross-sections arises when the CSM limiting stress f_{csm} exceeds the yield stress f_y . Consequently, for hot-finished structural carbon steel sections there are no strength benefits due to the strain hardening model adopted in Equation 3.27, while for the other considered materials, there is additional resistance with increasing deformation capacity.

For slender cross-sections, the axial compressive resistance can be determined using

$$N_{\text{csm}} = \frac{\varepsilon_{\text{csm}}}{\varepsilon_y} Af_y \text{ for } 0.3 < \bar{\lambda}_c \leq 0.6, \quad (3.40)$$

which is the yield load Af_y factored by the strain ratio.

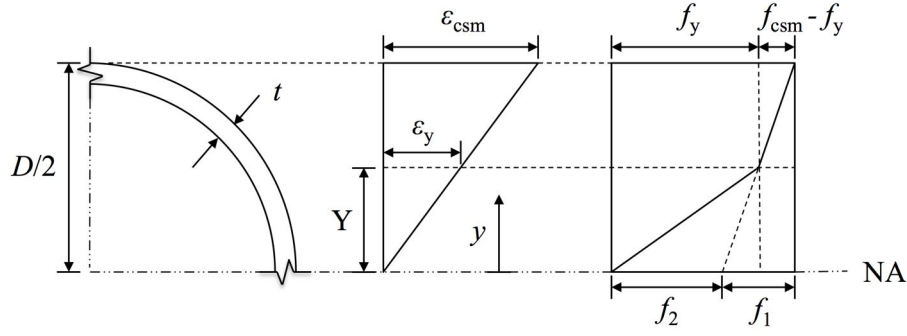


Figure 3.24: Strain and stress profiles for half of a CHS (a quarter of the cross-section is shown).

3.9.2 Bending resistance

The derivation of the CSM bending resistance function for non-slender CHS (Liew, 2014) is first described. In the derivation, it is assumed that plane sections remain plane and normal to the neutral axis during bending, and that the cross-section shape does not significantly distort before the outer-fibre strain ε_{csm} is attained. The corresponding linear strain and two-stage linear piecewise stress distributions (arising from the elastic, linear hardening material model) for half of a CHS are shown in Figure 3.24.

From Figure 3.24, the bending capacity of a cross-section can be expressed by

$$M_{\text{csm}} = W_{\text{pl}}f_{\text{csm}} - (W_{\text{pl}} - W_{\text{el}})f_1 - W_{\text{w}}f_2, \quad (3.41)$$

in terms of the elastic section modulus W_{el} , plastic section modulus W_{pl} , and the introduced modulus W_{w} . The term $W_{\text{w}}f_2$ represents the moment caused by the triangular shaped stress block associated with stress f_2 for $|y| \leq Y$. The first yield distance Y from the neutral axis NA is given by

$$Y = \frac{0.5D}{\varepsilon_{\text{csm}}/\varepsilon_{\text{y}}}. \quad (3.42)$$

Using the CSM material model (Equation 3.26 from Section 3.8) and the stress distribution geometry in Figure 3.24, stresses f_1 and f_2 are determined as

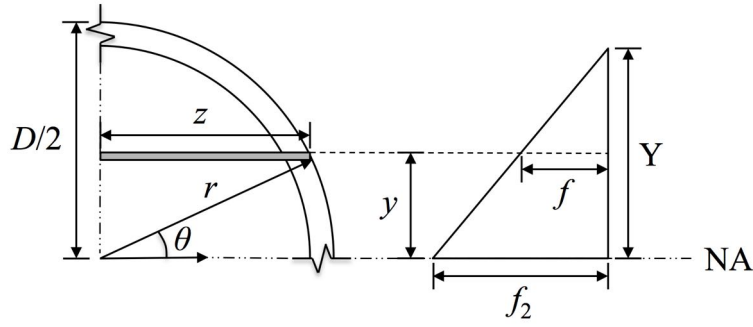


Figure 3.25: Geometry for the derivation of W_w for CHS.

$$\frac{f_1}{f_y} = \frac{E_{sh}}{E} \frac{\varepsilon_{csm}}{\varepsilon_y} \quad (3.43)$$

and

$$\frac{f_2}{f_y} = 1 - \frac{E_{sh}}{E}. \quad (3.44)$$

Normalising Equation 3.41 by the plastic moment capacity, $M_{pl} = W_{pl}f_y$, and substituting in the expressions for f_{csm} , f_1 and f_2 gives

$$\frac{M_{csm}}{M_{pl}} = 1 + \frac{E_{sh}}{E} \left(\frac{\varepsilon_{csm}}{\varepsilon_y} \frac{W_{el}}{W_{pl}} - 1 \right) - \frac{W_w}{W_{pl}} \left(1 - \frac{E_{sh}}{E} \right). \quad (3.45)$$

The moment $M_{f_2} = W_w f_2$ is defined by

$$M_{f_2} = W_w f_2 = \int_{A_Y} f y dA_Y = \int_{A_Y} f_2 g(y) y dA_Y, \quad (3.46)$$

from which the introduced modulus W_w can be determined using

$$W_w = \int_{A_Y} g(y) y dA_Y, \quad (3.47)$$

where f is a stress distribution and the function $g(y)$ represents the triangular stress distribution normalised by f_2 . The integral is only evaluated to the first yield point $|y| = Y$, and the associated area to integrate over is A_Y .

If integration is performed over one quarter of the cross-section, as in Figure 3.25,

then the result may be multiplied by four for the entire CHS to give the modulus W_w corresponding to the area A_Y . From the equation of a circle,

$$z^2 + y^2 = r^2 \quad (3.48)$$

with z and y co-ordinates and with radius r , the positive branch length can be calculated from

$$z = r \sqrt{1 - \frac{y^2}{r^2}}. \quad (3.49)$$

The introduced modulus $W_{w,r}$ for a solid circle of radius r can be determined from

$$W_{w,r} = 4 \int_0^Y y \left(1 - \frac{y}{Y}\right) z dy = 4r \int_0^Y y \sqrt{1 - \frac{y^2}{r^2}} dy - \frac{4r}{Y} \int_0^Y y^2 \sqrt{1 - \frac{y^2}{r^2}} dy \quad (3.50)$$

which is four times the integral of the stress triangle multiplied by the associated area and lever arm, where Y from Equation 3.42 is the value of y at first yield, which is constant for a given strain ratio. The solution to this integral is given by

$$W_{w,r} = \frac{4r^3}{3} \left[1 - \left(1 - \frac{Y^2}{r^2}\right)^{\frac{3}{2}} \right] - \frac{r}{Y} \left[\frac{Y}{2} (2Y^2 - r^2) \sqrt{1 - \frac{Y^2}{r^2}} + \frac{r^3}{2} \sin^{-1} \left(\frac{Y}{r} \right) \right]. \quad (3.51)$$

W_w for a circular hollow section can then be calculated from $W_w = W_{w,r_2} - W_{w,r_1}$, where W_{w,r_2} is evaluated for the outer radius $r_2 = D/2$, leading to

$$W_{w,r_2} = \frac{4r_2^3}{3} [1 - \cos^3 \alpha] - \frac{r_2^3}{\sin \alpha} \left[\sin^3 \alpha \cos \alpha + \frac{\alpha}{2} - \frac{\sin \alpha \cos \alpha}{2} \right] \quad (3.52)$$

and W_{w,r_1} is determined for the inner radius $r_1 = r_2 - t$ from

$$W_{w,r_1} = \frac{4r_1^3}{3} [1 - \cos^3 \beta] - \frac{r_1^3}{\sin \beta} \left[\sin^3 \beta \cos \beta + \frac{\beta}{2} - \frac{\sin \beta \cos \beta}{2} \right], \quad (3.53)$$

where $\sin \alpha = \frac{Y}{r_2}$, $\cos \alpha = \sqrt{1 - \frac{Y^2}{r_2^2}}$, $\sin \beta = \frac{Y}{r_1}$ and $\cos \beta = \sqrt{1 - \frac{Y^2}{r_1^2}}$. The analytical

formula is valid for $r_2/r_1 < \varepsilon_{\text{csm}}/\varepsilon_y$.

Substituting $W_w = W_{w,r_2} - W_{w,r_1}$ and the expressions for the two introduced moduli into Equation 3.45 leads to the exact analytical CSM bending resistance function. However, owing to the lengthy W_w term, the exact expression is not suitable for use in design, and therefore a simplified design equation is sought.

For a strain ratio $\varepsilon_{\text{csm}}/\varepsilon_y$ of unity, $f_{\text{csm}} = f_y$, and the introduced modulus W_w simplifies to

$$W_w = W_{\text{pl}} - W_{\text{el}}. \quad (3.54)$$

For strain ratios greater than unity, the introduced modulus W_w can be approximated by

$$W_w = (W_{\text{pl}} - W_{\text{el}}) / \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \right)^2 \quad (3.55)$$

from Gardner (2008), which can then be substituted into Equation 3.45 to give the normalised CSM moment capacity expressed by

$$\frac{M_{\text{csm}}}{M_{\text{pl}}} = 1 + \frac{E_{\text{sh}}}{E} \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \frac{W_{\text{el}}}{W_{\text{pl}}} - 1 \right) - \left(1 - \frac{W_{\text{el}}}{W_{\text{pl}}} \right) \left(1 - \frac{E_{\text{sh}}}{E} \right) / \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \right)^2. \quad (3.56)$$

By noting that in general $E_{\text{sh}}/E \ll 1$, the $(1 - E_{\text{sh}}/E)$ term can conservatively be taken as unity as the final term is subtractive, while the remaining expression can be forced through M_{el} at a strain ratio of unity to give the simplified M_{csm} equation,

$$M_{\text{csm}} = M_{\text{pl}} \left[1 + \frac{E_{\text{sh}}}{E} \frac{W_{\text{el}}}{W_{\text{pl}}} \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} - 1 \right) - \left(1 - \frac{W_{\text{el}}}{W_{\text{pl}}} \right) / \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \right)^2 \right] \text{ for } \bar{\lambda}_c \leq 0.3. \quad (3.57)$$

Design equations of this format have also been used for determining the CSM bending resistance of I-sections and box sections (Gardner et al., 2011b). The exact analytical CSM bending resistance expression, Equation 3.45, is plotted along with the simplified CSM design expression, Equation 3.57, in Figure 3.26 for a typical ratio of E_{sh}/E of

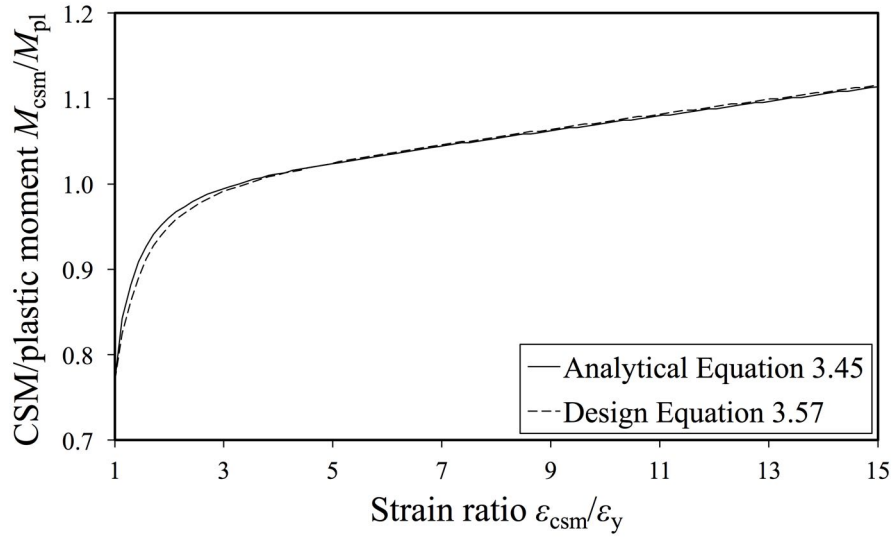


Figure 3.26: Comparison between the exact and approximate (design) CSM bending capacity predictions $M_{\text{csm}}/M_{\text{pl}}$ with varying $\epsilon_{\text{csm}}/\epsilon_y$.

1/100. The design equation can be seen to tend towards the analytical expression for the higher strain ratios where W_w has a smaller influence, and that, overall, the difference between the two curves is minimal.

The variation in bending capacity $M_{\text{csm}}/M_{\text{pl}}$ with strain ratio $\epsilon_{\text{csm}}/\epsilon_y$ for various strain hardening ratios E_{sh}/E is plotted in Figure 3.27. The bending capacity at a strain ratio of unity is the elastic moment M_{el} . The subsequent increase in bending capacity for $\epsilon_{\text{csm}}/\epsilon_y > 1$ is dependent upon the strain hardening ratio, where a larger strain hardening modulus leads to a greater increase in bending capacity with increasing strain ratio $\epsilon_{\text{csm}}/\epsilon_y$. If the strain hardening modulus is taken as zero, the bending capacity is asymptotic to the plastic moment capacity. It should be noted that the CSM bending resistance function for CHS is sensitive to the strain ratio in the region of $\epsilon_{\text{csm}}/\epsilon_y < 3$, reflecting the relatively high shape factor of CHS and the marked increases in moment capacity that arise during the initial spread of plasticity.

For slender cross-sections the CSM bending resistance M_{csm} can be calculated by factoring the elastic moment capacity $W_{\text{el}}f_y$ by the strain ratio,

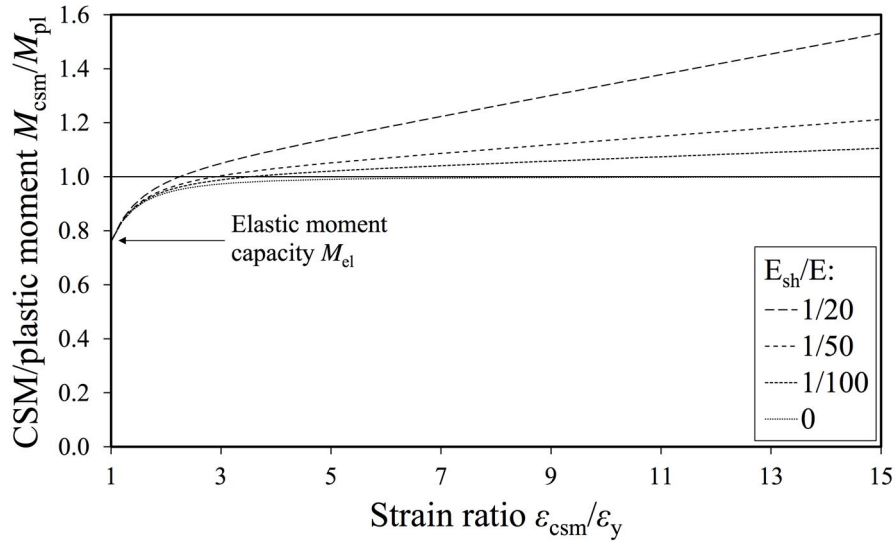


Figure 3.27: The influence of E_{sh}/E on M_{csm}/M_{pl} with varying $\varepsilon_{csm}/\varepsilon_y$.

$$M_{csm} = \frac{\varepsilon_{csm}}{\varepsilon_y} W_{el} f_y \text{ for } 0.3 < \bar{\lambda}_c \leq 0.6. \quad (3.58)$$

3.10 Comparison with test data and existing design methods

The CSM predictions for compression and bending resistances have been compared with the measured ultimate values from the collected experiments. The average ultimate test loads N_u and moments M_u normalised by the CSM (N_{csm} , M_{csm}) and Eurocode (N_{EC} , M_{EC}) predictions have been determined for each material type and are summarised in Tables 3.10 and 3.12 respectively for non-slender cross-sections ($\bar{\lambda}_c \leq 0.3$), and in Tables 3.11 and 3.13 for slender cross-sections ($\bar{\lambda}_c > 0.3$). EN 1993-1-1 (2014) and EN 1993-1-4 (2015) do not provide design expressions for slender CHS in compression or bending; the design formulae for slender cross-sections from BS 5950-1 (2000) with updated Eurocode class 3 limits have therefore been utilised, as discussed previously in Section 3.4.6. EN 1999-1-1 (2007) provides design expressions for slender class 4 aluminium CHS resistances in compression and bending. The coefficients of variation (COV) have also been calculated to quantify and allow comparisons of the scatter of the

Table 3.10: CSM and Eurocode compression resistance prediction comparison for $\bar{\lambda}_c \leq 0.3$.

Material	Mean		COV	
	N_u/N_{csm}	N_u/N_{EC}	N_u/N_{csm}	N_u/N_{EC}
Hot-finished structural steel	1.08	1.08	0.12	0.12
Very high strength structural steel	1.19	1.21	- ¹	- ¹
Cold-formed structural steel	1.12	1.17	0.17	0.20
Stainless steel	1.18	1.25	0.12	0.16
Aluminium	1.09	1.13	0.08	0.08
Average	1.14	1.19	0.14	0.18

¹ Insufficient experimental data

Table 3.11: CSM and Eurocode compression resistance prediction comparison for $0.3 < \bar{\lambda}_c \leq 0.6$.

Material	Mean		COV	
	N_u/N_{csm}	N_u/N_{EC}	N_u/N_{csm}	N_u/N_{EC}
Hot-finished structural steel	- ¹	- ¹	- ¹	- ¹
Very high strength structural steel	1.20	1.50	0.02	0.13
Cold-formed structural steel	1.04	1.27	0.10	0.14
Stainless steel	1.14	1.31	0.12	0.11
Aluminium	1.05	1.06	0.09	0.09
Average	1.10	1.31	0.11	0.15

¹ Insufficient experimental data

predictions. The ultimate experimental loads normalised by their CSM and Eurocode predictions ($N_u/N_{u,pred}$, $M_u/M_{u,pred}$) have been plotted for compression in Figure 3.28 and bending in Figure 3.29.

Overall, on average, for both non-slender and slender cross-sections the CSM predicts cross-section compression and bending resistances that are more accurate and consistent compared with those from the Eurocodes. Considering initially the capacity predictions for the non-slender cross-sections, apart from for hot-finished structural carbon steel, the CSM predictions are on average closer to the measured ultimate resistance than those from the Eurocodes and exhibit reduced scatter. For hot-finished structural carbon steel, the CSM and EN 1993-1-1 (2014) predictions are the same for cross-sections in compression (due to the use of $E_{sh} = 0$), while in bending the CSM is more conservative than EN 1993-1-1 (2014) as the predicted capacity does not reach

Table 3.12: CSM and Eurocode bending resistance prediction comparison for $\bar{\lambda}_c \leq 0.3$.

Material	Mean		COV	
	M_u/M_{csm}	M_u/M_{EC}	M_u/M_{csm}	M_u/M_{EC}
Hot-finished structural steel	1.06	1.05	0.11	0.12
Very high strength structural steel	1.47	1.49	¹	¹
Cold-formed structural steel	1.10	1.18	0.10	0.12
Stainless steel	1.15	1.24	0.01	0.09
Aluminium	1.19	1.36	0.03	0.15
Average	1.11	1.16	0.11	0.15

¹ Insufficient experimental data

Table 3.13: CSM and Eurocode bending resistance prediction comparison for $0.3 < \bar{\lambda}_c \leq 0.6$.

Material	Mean		COV	
	M_u/M_{csm}	M_u/M_{EC}	M_u/M_{csm}	M_u/M_{EC}
Hot-finished structural steel	¹	¹	¹	¹
Very high strength structural steel	1.50	1.63	0.05	0.04
Cold-formed structural steel	1.25	1.33	0.15	0.14
Stainless steel	1.26	1.21	0.09	0.10
Aluminium	1.39	1.39	0.04	0.03
Average	1.32	1.38	0.13	0.15

¹ Insufficient experimental data

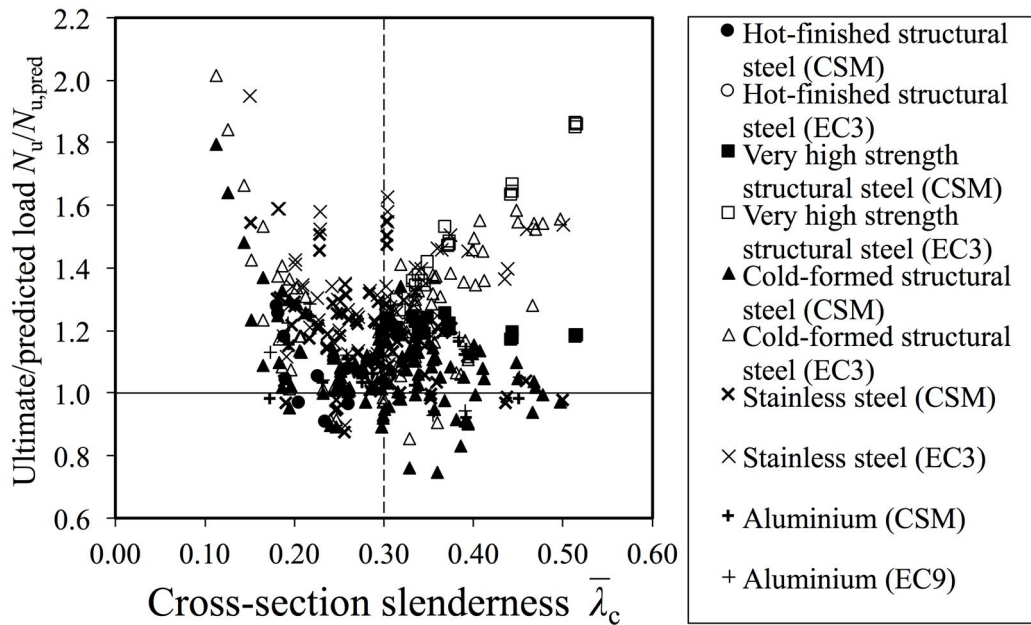


Figure 3.28: CSM and Eurocode compression resistance prediction comparison.

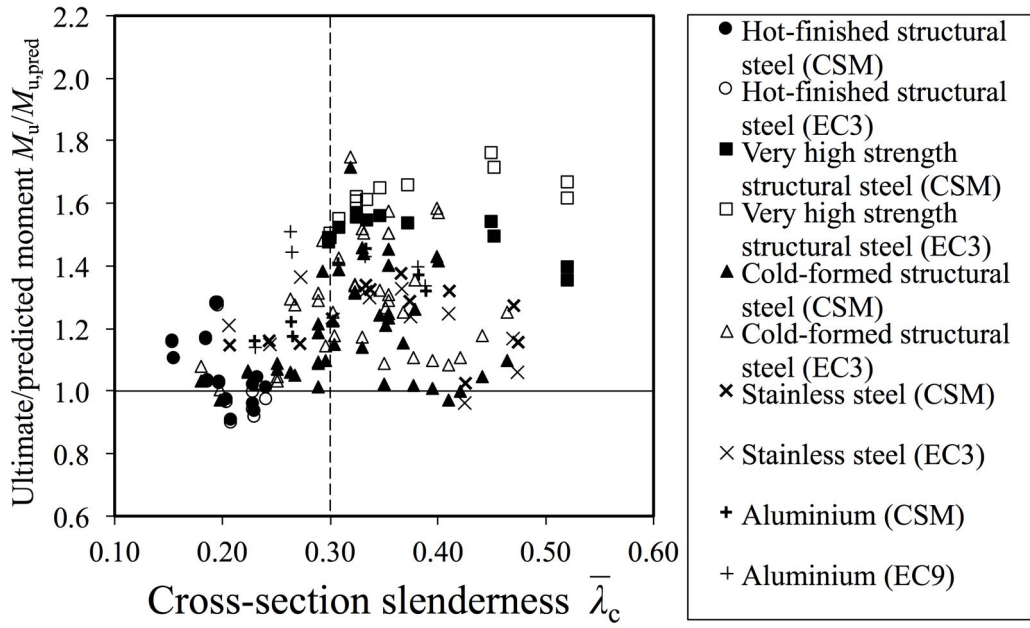


Figure 3.29: CSM and Eurocode bending resistance prediction comparison.

M_{pl} and only approaches it asymptotically. Inspection of Figure 3.29 reveals that this increased conservatism is often beneficial as a number of hot-finished bending results are overpredicted by EN 1993-1-1 (2014) and are better predicted by the lower CSM resistance.

For the slender CHS in compression, the CSM predictions are on average more accurate than those from the Eurocodes for all metallic materials. In terms of consistency in the compression predictions, there is reduced scatter for very high strength structural carbon steel and cold-formed structural carbon steel. The COV value is unchanged for aluminium and slightly higher for stainless steel. Considering slender CHS in bending, the CSM predictions are the same or more accurate than the Eurocodes for all materials apart for stainless steel. The latter outcome is due to the EN 1993-1-4 (2015) class 3 bending limit for stainless steel being much higher than those in EN 1993-1-1 (2014) and EN 1999-1-1 (2007), as apparent in Figure 3.20. This leads to a long $M_{u,pred} = M_{el}$ plateau that extends far beyond the equivalent structural carbon steel and aluminium class 3 bending limits and the CSM yield slenderness limit, resulting in the CSM bending resistance predictions for slender stainless steel CHS being more conservative than those

from EN 1993-1-4 (2015). This issue is essentially a consequence of comparing a single harmonised (CSM) design approach against multiple design standards that have varying class 3 limits in compression and bending and between material types, and also differing reduction factors for local buckling in slender cross-sections.

Overall, the improved capacity predictions of between about 5% and 20% on average for the different materials, together with the reduced scatter, are clearly evident in Figures 3.28 and 3.29 and Tables 3.10 to 3.13.

3.11 Conclusions

A comprehensive structural carbon steel, stainless steel and aluminium CHS dataset has been collected from the literature for pure compression and bending. These existing results have been supplemented with ten additional stainless steel CHS stub columns, with the tensile material properties determined through tensile coupon tests. The CSM has been extended to cover the design of CHS and has been seen to provide improved cross-section resistance predictions of between about 5% and 20% for different metallic materials over traditional design methods. For non-slender CHS ($\bar{\lambda}_c \leq 0.3$) the CSM is generally more accurate and consistent in its capacity predictions and for slender CHS it also offers improved predictions of cross-section resistance, although with more scatter in some cases. The results for slender CHS ($\bar{\lambda}_c > 0.3$) are more varied primarily due to the differing, bespoke treatments across the various materials in existing design methods, compared to the standard, harmonised approach across all material types for the CSM. Further work is currently underway in refining the material models, with the aim to gain improved predictions over those currently attained, and undertaking reliability analyses. Improved predictions of CHS cross-section resistances will lead to lighter structures with more efficient material use, leading to more sustainable construction.

Chapter 4

Flexural buckling of CHS compression members

4.1 Introduction

EN 1993-1-4 (2015) provides design guidance for the flexural buckling capacity of stainless steel CHS members through a harmonised approach that is consistent with the design of carbon steel elements in EN 1993-1-1 (2014). There are however two issues with the current flexural buckling design approach in EN 1993-1-4 (2015). Firstly, there is a very limited existing stainless steel compressive CHS dataset with a high degree of scatter, which consists primarily of austenitic stub column results from Rasmussen and Hancock (1993a), Talja (1997), Burgan et al. (2000), Rasmussen (2000), Young and Hartono (2002), Kuwamura (2003), Gardner and Nethercot (2004a), Lam and Gardner (2008), Uy et al. (2011) and Zhao et al. (2015a) and a small number of longer member results from Rasmussen and Hancock (1993a), Talja (1997), Burgan et al. (2000), Young and Hartono (2002) and Zhao et al. (2016c). There are also a limited number of duplex stub column tests by Bardi and Kyriakides (2006), Paquette and Kyriakides (2006) and Lam and Gardner (2008) and ferritic stub column tests by Stangenberg (2000), but no existing duplex or ferritic longer column compressive test results. The second issue is that it has been observed that the EN 1993-1-4 (2015)

flexural buckling curve is unconservative for the existing dataset, which can result in unsafe design (Rasmussen and Rondal, 2000; Young and Hartono, 2002; Young and Ellobody, 2006; Ellobody and Young, 2007; Ashraf et al., 2008a; Theofanous et al., 2009b; Shu et al., 2014). The reason for this unconservative buckling curve is that it was calibrated using predominantly square hollow section (SHS) and rectangular hollow section (RHS) column buckling test results (Rasmussen and Rondal, 2000), due to a lack of stainless steel CHS experimental data at the time the standard was produced.

Consequently, there is a need firstly to expand the existing stainless steel CHS compressive flexural buckling dataset and secondly to use this newly expanded dataset to evaluate existing proposals for an updated buckling curve and to put forward design recommendations for a new safe, but still efficient, stainless steel CHS buckling curve, which is the focus of the work presented in this chapter. The experimental programme has been published previously by Buchanan et al. (2016c) and Buchanan et al. (2016d).

4.2 Flexural buckling tests

4.2.1 General overview

The experimental programme consisted of material property tests, cross-section level stub column tests and member level concentrically loaded, pin-ended flexural buckling column tests. The test specimens covered a wide range of local slenderness values, with five different cross-sections in total covering classes 1 to 4, a wide range of global slenderness values, with effective column lengths varying from 300 mm to 3000 mm, and included all three main types of stainless steel used in construction. The experimental programme was split between Imperial College London (ICL) and Universitat Politècnica de Catalunya (UPC). The column tests for the two austenitic (A) cross-sections, 106×3 CHS (class 1) and 104×2 CHS (class 3), and the duplex (D) cross-section, 88.9×2.6 CHS (class 4), were carried out at ICL, along with the tensile coupon testing. The column tests

on the two ferritic (F) cross-sections, 80×1.5 CHS (class 3) and 101.6×1.5 CHS (class 4), were undertaken at UPC. The ferritic pin-ended columns had 10 mm thick end plates welded to each end to allow mechanical attachment to the knife edges. The specimen notation can be explained by means of an example, e.g. specimen 104 $\times$ 2-400-P is the 104 $\times$ 2 CHS cross-section with an effective length of 400 mm and ‘P’ pin-ended conditions, and a specimen ending with ‘R’ denotes a repeat specimen.

4.2.2 Material properties

Tensile coupon tests were undertaken to determine the basic material stress–strain properties. The tensile coupon tests and their results were presented in Section 3.4.3.

4.2.3 Geometric properties

The geometric properties of the specimens were carefully measured before testing. Outer diameter measurements were taken at three equally spaced longitudinal locations for the shorter pin-ended columns ($L < 400$ mm) and at five equally spaced longitudinal locations for longer columns. At each location the outer diameter was recorded in four evenly distributed orientations (at 45° intervals) with callipers, allowing the average outer diameter D of the specimen to be calculated along with its ‘out-of-roundness’ as defined in EN 10219-2 (2006a), and reproduced as Equation 3.3 in Chapter 3. The maximum ‘out-of-roundness’ O of the non-repeat specimens was 2.0%, satisfying the limit of $O \leq 2.0\%$, although the 106 $\times$ 3-750-PR repeat specimen had $O = 2.4\%$. The average wall thickness t was determined from measurements of the wall thickness at the ends of the tubes at eight equally spaced locations (at 45° intervals) around the circumference, at a distance not less than $2t$ from the weld location in accordance with EN 10219-2 (2006a). The wall thicknesses of the pin-ended ferritic columns could not be measured due to the welded end plates, although the variation around the ferritic stub columns was small ($< 4.5\%$, < 0.06 mm) and the average wall thickness t for the

ferritic stub columns is used for the pin-ended columns. The average specimen length L , which includes the additional length from the knife edges, was calculated by averaging the specimen length from four separate measurements taken at 90° locations around the circumference. Lengths were measured on four faces using callipers for the short to intermediate length specimens ($L \leq 600$ mm) and for longer specimens a tape measure was used. The mid-point global imperfections were recorded by measuring the deviation from a flat plane linking the two ends of the specimens. For the shorter specimens the deviation was measured using the bottom of a spirit level and feeler gauges, whereas for longer specimens at ICL a self levelling laser and steel rule were used and at UPC an aluminium extrusion was placed over the end plates and the deviation measured using callipers. The imperfection measurements were made on the two faces parallel to the knife-edge axis (i.e. to obtain the imperfection in the direction of buckling) for the pin-ended specimens. The average geometric properties of the specimens are provided in Tables 4.1 to 4.5.

The weld position for the UPC specimens was constant, with the weld orientated to be on the least compressed face. For the ICL tests the specimens were orientated with the smallest average diameter to be perpendicular to the knife edge axis (i.e. the orientation with the minimum second moment of area I in the direction of buckling) which had the consequence that the weld position rotated between specimens.

4.2.4 Stub column tests

Stub column tests were undertaken to determine the cross-sectional load carrying capacity under pure compression and also to ascertain compressive material properties. The stub column tests have been discussed previously in Section 3.4.

4.2.5 Flexural buckling test setup

The flexural buckling tests consisted of 17 austenitic, 9 duplex and 11 ferritic concentrically loaded, pin-ended column specimens with a wide range of local and global slendernesses. The specimen lengths were chosen to provide a range of global slenderness $\bar{\lambda}$ values (calculated using Equations 3.6 and 3.7 from Chapter 3 and reported in Tables 4.1 to 4.5), with the maximum length dictated by the testing machines used.

Three different experimental setups were utilised due to testing in different locations and certain setups having specimen length limitations. The Instron 2000kN-SPL test setup used for the austenitic and duplex columns at ICL is shown in Figure 4.1. Two setups were used at UPC for the ferritic columns — specimens with $L \leq 1100$ mm were tested using an Instron 8805 testing machine as shown in Figure 4.2, while the 1600 mm specimens were tested using an Ibertest MDA-700 setup as shown in Figure 4.3. The lengths L reported for the pin-ended columns are the effective lengths and include the additional length from the knife edges, which were each 77 mm thick at ICL and 40 mm thick at UPC, and the 10 mm thick welded end plates at UPC.

A spherical head was used at UPC with the Instron 8805 test setup as shown in Figure 4.2, to attain full contact with the welded end plates. Prior to testing the spherical head was loosened and checked for freedom of movement; the specimen was then bolted in place and an installation load of 5 kN was applied at which point the bolts were tightened, locking the spherical head in place, before removing the 5 kN installation load. The spherical head could not be used with the Ibertest setup due to limited headroom and was not used with the ICL tests as the column ends were machined to be flat and parallel prior to testing (see Section 3.4.2).

A range of instrumentation was used throughout the testing programme depending upon the testing machines and setups. The end rotations were measured at both ends of the specimens using inclinometers. Two electrical resistance strain gauges were affixed onto

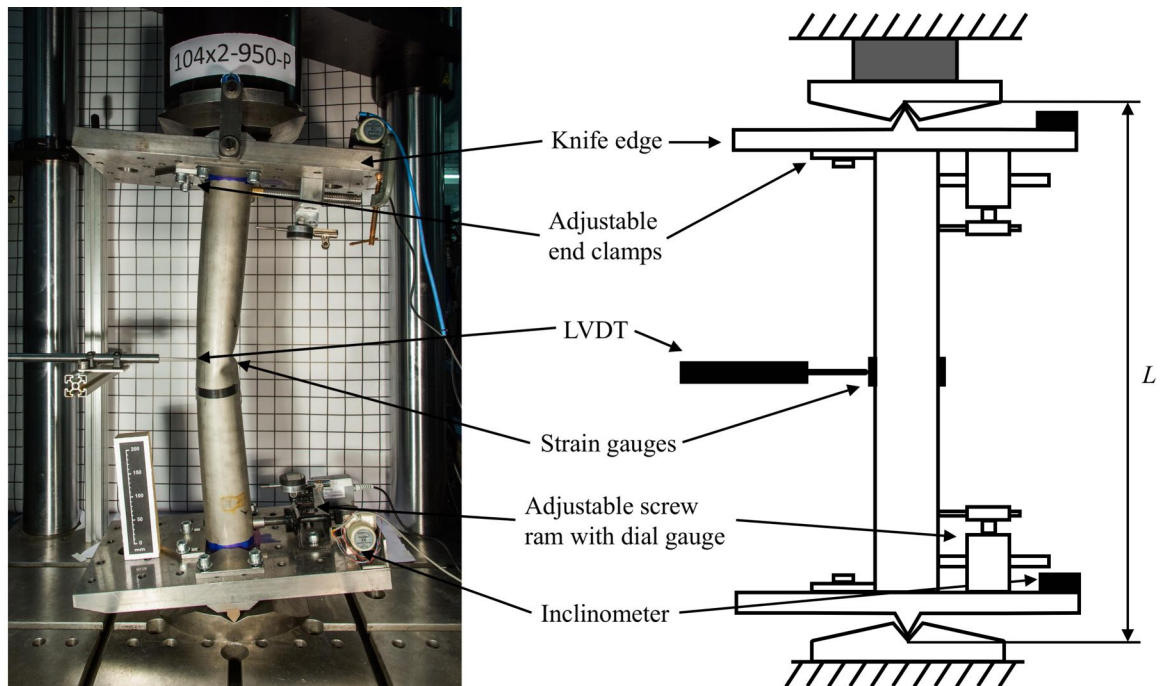


Figure 4.1: The ICL Instron 2000kN-SPL pin-ended column test setup.

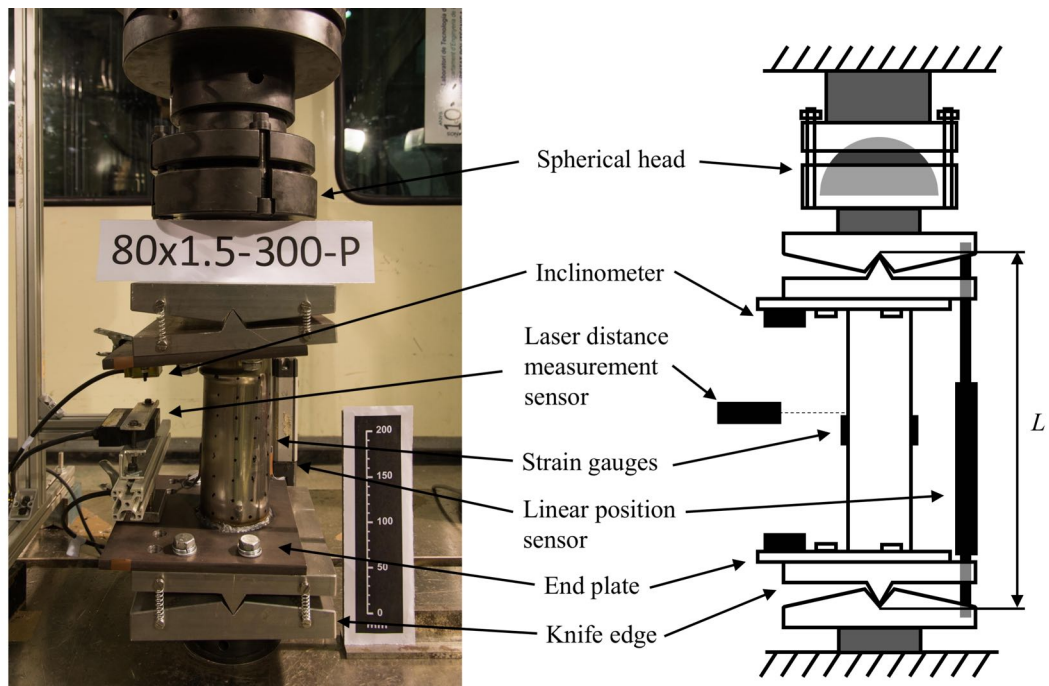


Figure 4.2: The UPC Instron 8805 pin-ended short column test setup.

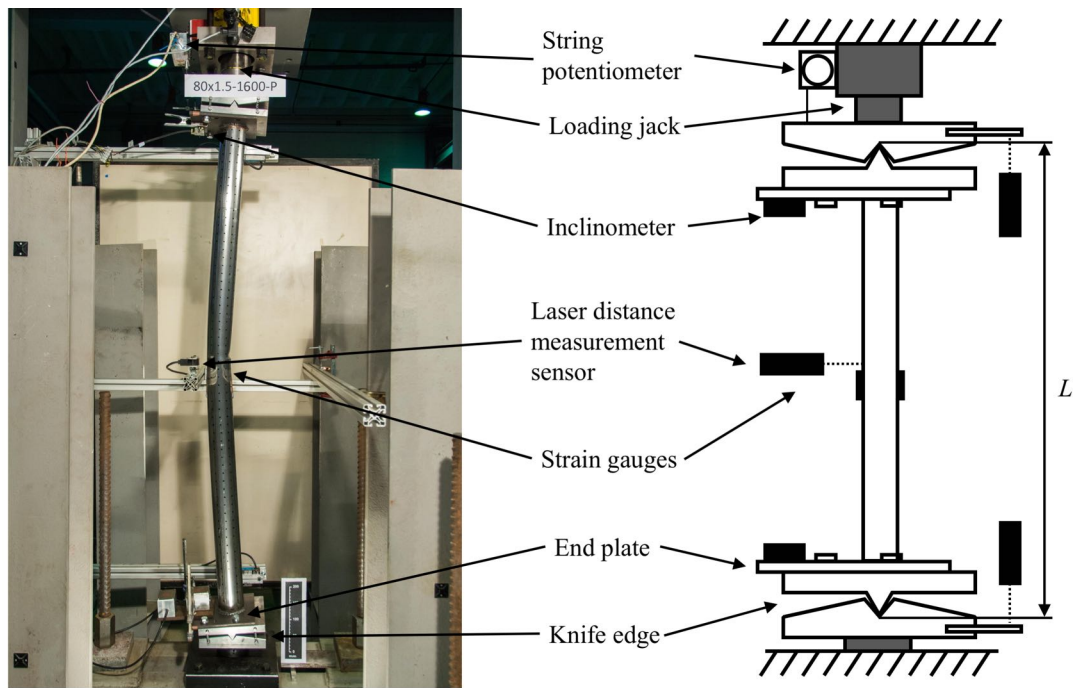


Figure 4.3: The UPC Ibertest MDA-700 pin-ended long column test setup.

opposite faces at mid-height to measure the maximum and minimum strains arising from the flexural buckling. The mid-height lateral deflection was measured at UPC with a laser distance measurement sensor (shown in Figure 4.4) and at ICL using a linear variable differential transformer (LVDT). For the Instron testing machines the load was measured using a load cell within the testing machines, whereas for the Ibertest testing machine the applied load was calculated from the pressure within the loading jack. For the end shortening, the position of the top platen was measured using an internal LVDT for the Instron testing machines and in the Ibertest setup using a string potentiometer attached to and controlling the displacement of the loading jack. At UPC a second sensor was used to measure the end shortening — for the Instron setup a linear position sensor was attached between the outer non-rotating female parts of the knife edges and for the Ibertest setup two additional laser measurement devices were similarly utilised. The parameters being measured were recorded at 2 Hz using a HBM MGCplus datalogger and Catman software at UPC and a proprietary in-house developed datalogger and recording software at ICL. All of the tests were also filmed for later review, as shown in Figure 4.5.



Figure 4.4: The laser distance measurement sensor at UPC.

4.2.6 Global imperfection and applied eccentricity

In line with Rasmussen and Hancock (1993a) a combined equivalent global imperfection and applied eccentricity of $L/1000$ was applied to all of the specimens to simulate the typically assumed imperfections in real columns. The combined initial imperfection ω_0 and eccentricity e_0 can be determined from

$$\omega_0 + e_0 = \frac{EI (\varepsilon_{\max} - \varepsilon_{\min})}{DN} - \omega, \quad (4.1)$$

where E is the Young's modulus, I is the second moment of area, N is the axial load, D is the average outer diameter, ω is the mid-height lateral displacement and $\varepsilon_{\max}$ and $\varepsilon_{\min}$ are the maximum and minimum strains from the two extreme fibres of the cross-sections (Zhao et al., 2015a; Huang and Young, 2014; Zhao et al., 2015c).

The eccentricity was calculated by placing the specimen in the centre of the knife edges and then loading at a displacement rate of 0.2 mm/min to approximately 15% of the



Figure 4.5: A flexural buckling test being filmed for later review.

expected failure load N_u , ensuring that the specimen remained elastic but had sufficient load to have strains and lateral displacement that could be reliably measured. For the Instron testing machines a testing program was created, which is outlined as follows:

1. 10 second pause at the start — which prevents the testing machine jack from moving immediately when the test program is started,
2. the hydraulic jack then moves at -0.2 mm/min to a maximum displacement of -10 mm relative to the starting position, while the operator has an override button to stop the jack when 15% of N_u is attained,
3. a 60 second hold then occurs, which keeps the specimen at 15% of N_u for a consistent length of time,
4. the jack then moves at a rate of +0.2 mm/min to a height +1 mm relative to the original starting position, unloading the specimen, with an operator override to stop the unloading process with a small amount of load (approximately 5 kN) still applied to prevent the specimen from moving.

The testing program reduced the risk of operator error, such as incorrectly typing in the displacement rate, and ensured a consistent process between eccentricity measuring runs. Following on from the first check the specimen was then carefully moved across the knife edges to attain the $L/1000$ eccentricity; at ICL, an adjustable screw ram with a dial gauge was used at both ends of the columns to allow very controlled small movements of the column ends (shown in Figure 4.6) in conjunction with monitoring the amount of mid-height movement with the LVDT. At UPC, the specimens were moved using a light tap from a hammer onto the end plates and careful monitoring of the mid-height movement using the laser distance sensor; the specimens could be moved laterally a small amount due to the end plates having oversized bolt holes (as visible in Figure 4.7). The specimen would then be loaded again to 15% of N_u and its position further adjusted until a satisfactory eccentricity was attained. The calculated combined global imperfection and eccentricity $L/(\omega_0 + e_0)$ from Equation 4.1, along with the

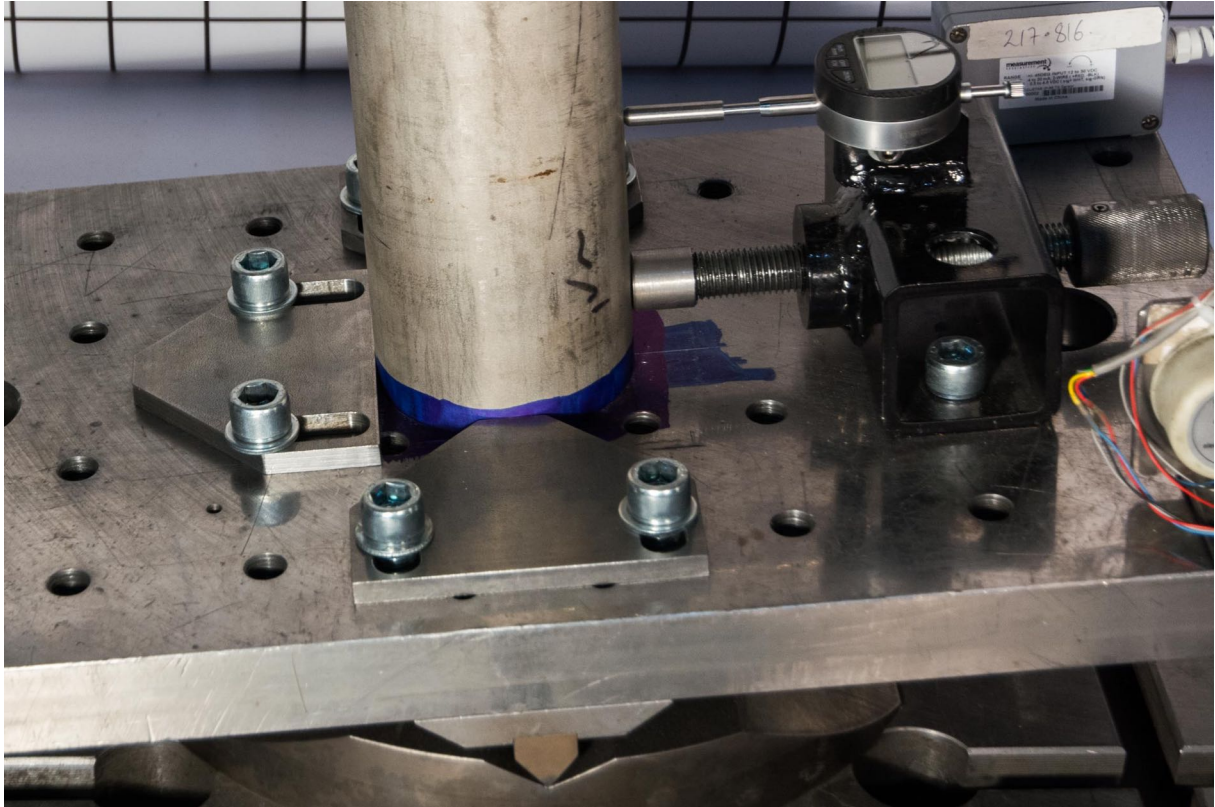


Figure 4.6: An adjustable screw ram with a dial gauge at ICL.

measured global imperfection L/ω_0 are reported in Tables 4.1 to 4.5. It should be noted that while this method was found to reflect the physical movement (movement in one direction would result in an increase of the calculated eccentricity in the same direction), it was observed to be very sensitive. There would be small variations in the average eccentricity depending on the displacement rate of the top platen, and even between runs there would be small differences without the specimen being moved or touched, which for the smallest eccentricities increased the error; this is reflected in Tables 4.1 to 4.5. Once the required eccentricity was attained, a displacement rate of 0.2 mm/min was used to load the specimens to their ultimate load, with the end of the test at UPC dictated by the maximum rotation of the knife edge (approximately 4°); the ICL tests were stopped at a similar maximum rotation.



Figure 4.7: Oversized bolt holes in an end plate at UPC.

4.2.7 Flexural buckling test results

The ultimate test loads N_u are reported in Tables 4.1 to 4.5, along with the location of manufacture for the 104×2 CHS specimens. The 106×3-750-P CHS specimen was repeated as it had a lower ultimate load than the shorter 106×3-550-P CHS specimen, although the remaining 106×3 CHS tube stock had an ‘out-of-roundness’ outside the $O \leq 2\%$ limit in EN 10219-2 (2006a) and the repeat test achieved a similar ultimate load to the original specimen.

The most common failure mode was global buckling, though the shorter specimens (CHS specimens shorter than and including 104×2-1650-P, 106×3-950-P and 88.9×2.6-950-P) also developed a mid-height local buckle on the compressed face after the peak load. The ferritic CHS specimens also exhibited global buckling failure modes, with a local buckle forming after the peak load. The very short ferritic CHS specimens (80×1.5-300-P, 101.6×1.5-350-P and 101.6×1.5-500-P) did not show an obvious global buckle at the

Table 4.1: Measured geometric properties and test results of the austenitic 106×3 CHS pin-ended columns.

Specimen	D (mm)	t (mm)	L (mm)	$\bar{\lambda}$	L/ω_0	$L/(\omega_0 + e_0)$	N_u (kN)
106×3-550-P	105.74	2.78	554.27	0.17	2217	1218	267.0
106×3-750-P	105.81	2.88	754.21	0.23	- ¹	1034	244.8
106×3-750-PR	105.77	2.72	754.21	0.23	1886	880	242.2
106×3-950-P	105.78	2.79	954.0	0.29	4770	957	253.4
106×3-1150-P	105.84	2.83	1154.0	0.36	4616	1000	248.8
106×3-1650-P	105.63	2.74	1657.0	0.51	1657	945	201.3
106×3-2150-P	105.88	2.71	2152.9	0.66	2871	997	185.5
106×3-2650-P	105.64	2.73	2652.5	0.82	5305	987	159.4
106×3-3080-P	105.67	2.70	3083.0	0.95	561	1044	150.8

¹ Initial imperfection less than 0.05 mm in buckling plane

Table 4.2: Measured geometric properties and test results of the austenitic 104×2 CHS pin-ended columns.

Specimen	Origin	D (mm)	t (mm)	L (mm)	$\bar{\lambda}$	L/ω_0	$L/(\omega_0 + e_0)$	N_u (kN)
104×2-550-P	FIN	103.97	1.89	553.77	0.20	- ¹	1180	241.1
104×2-750-P	FIN	104.01	1.89	753.84	0.28	5026	887	232.1
104×2-950-P	FIN	103.97	1.88	954.0	0.35	6360	827	204.2
104×2-1150-P	SWE	104.14	1.86	1153.5	0.42	577	1115	180.8
104×2-1650-P	SWE	104.09	1.85	1656.6	0.61	- ¹	963	154.8
104×2-2150-P	SWE	104.08	1.79	2153.6	0.79	2872	999	126.4
104×2-2650-P	FIN	103.92	1.75	2653.5	0.97	10614	1064	109.0
104×2-3080-P	SWE	104.10	1.79	3084.0	1.13	1542	1036	89.7

¹ Initial imperfection less than 0.05 mm in buckling plane

Table 4.3: Measured geometric properties and test results of the duplex 88.9×2.6 CHS pin-ended columns.

Specimen	D (mm)	t (mm)	L (mm)	$\bar{\lambda}$	L/ω_0	$L/(\omega_0 + e_0)$	N_u (kN)
88.9×2.6-400-P	88.63	2.37	403.90	0.23	2020	824	425.2
88.9×2.6-550-P	88.63	2.35	553.83	0.31	1846	1309	404.6
88.9×2.6-750-P	88.78	2.41	753.93	0.43	15078	944	389.6
88.9×2.6-950-P	88.77	2.37	954.0	0.54	6360	1075	344.4
88.9×2.6-1150-P	88.77	2.37	1154.0	0.65	23080	1142	295.3
88.9×2.6-1650-P	88.63	2.35	1656.6	0.93	1657	1022	243.4
88.9×2.6-2150-P	88.77	2.30	2152.8	1.20	783	716	164.7
88.9×2.6-2650-P	88.72	2.33	2653.4	1.49	1327	993	126.4
88.9×2.6-3080-P	88.67	2.32	3082.5	1.73	2466	1027	100.5

Table 4.4: Measured geometric properties and test results of the ferritic 80×1.5 CHS pin-ended columns.

Specimen	D (mm)	t (mm)	L (mm)	$\bar{\lambda}$	L/ω_0	$L/(\omega_0 + e_0)$	N_u (kN)
80×1.5-300-P	80.00	1.34	299.4	0.14	286	1270	126.1
80×1.5-450-P	80.01	1.34	448.9	0.21	1645	1290	119.4
80×1.5-700-P	80.04	1.34	698.6	0.33	8454	1151	111.1
80×1.5-900-P	80.03	1.34	899.1	0.42	1702	1068	105.8
80×1.5-1100-P	80.13	1.34	1099.1	0.51	3997	1109	97.8
80×1.5-1600-P	80.00	1.34	1599.3	0.75	2076	1019	77.9

Table 4.5: Measured geometric properties and test results of the ferritic 101.6×1.5 CHS pin-ended columns.

Specimen	D (mm)	t (mm)	L (mm)	$\bar{\lambda}$	L/ω_0	$L/(\omega_0 + e_0)$	N_u (kN)
101.6×1.5-350-P	101.66	1.34	349.4	0.12	5277	917	148.6
101.6×1.5-500-P	101.68	1.34	499.6	0.17	4157	1183	145.4
101.6×1.5-800-P	101.70	1.34	799.3	0.27	2686	1180	137.5
101.6×1.5-1100-P	101.82	1.34	1099.5	0.37	1588	1079	121.2
101.6×1.5-1600-P	101.71	1.34	1599.3	0.54	1905	1124	104.1

peak load and instead went on to develop ‘elephant’s foot’ local buckles close to one end of the specimen, an example of which is shown in Figure 4.2, which is more typical of a slender CHS stub column failure mechanism. The deformed specimens are shown in Figures 4.8 to 4.12. Strain visualization dots were painted onto the ferritic specimens due to the highly reflective nature of the surface, and post-test reviewing of the footage filmed showed Poisson’s ratio effects in the very short columns, with the painted dots moving radially outwards as the specimen was initially compressed. This radial expansion and elephant’s foot failure mode has been previously observed in finite element modelling of stainless steel elliptical hollow sections (EHS) (Theofanous et al., 2009b).

4.3 Finite element modelling

The finite element analysis package Abaqus/CAE 2016 (Dassault Systèmes Simulia Corp., 2015) has been used to produce additional structural performance data. A series of geometrically and materially nonlinear analyses with imperfections (GMNIA) have been

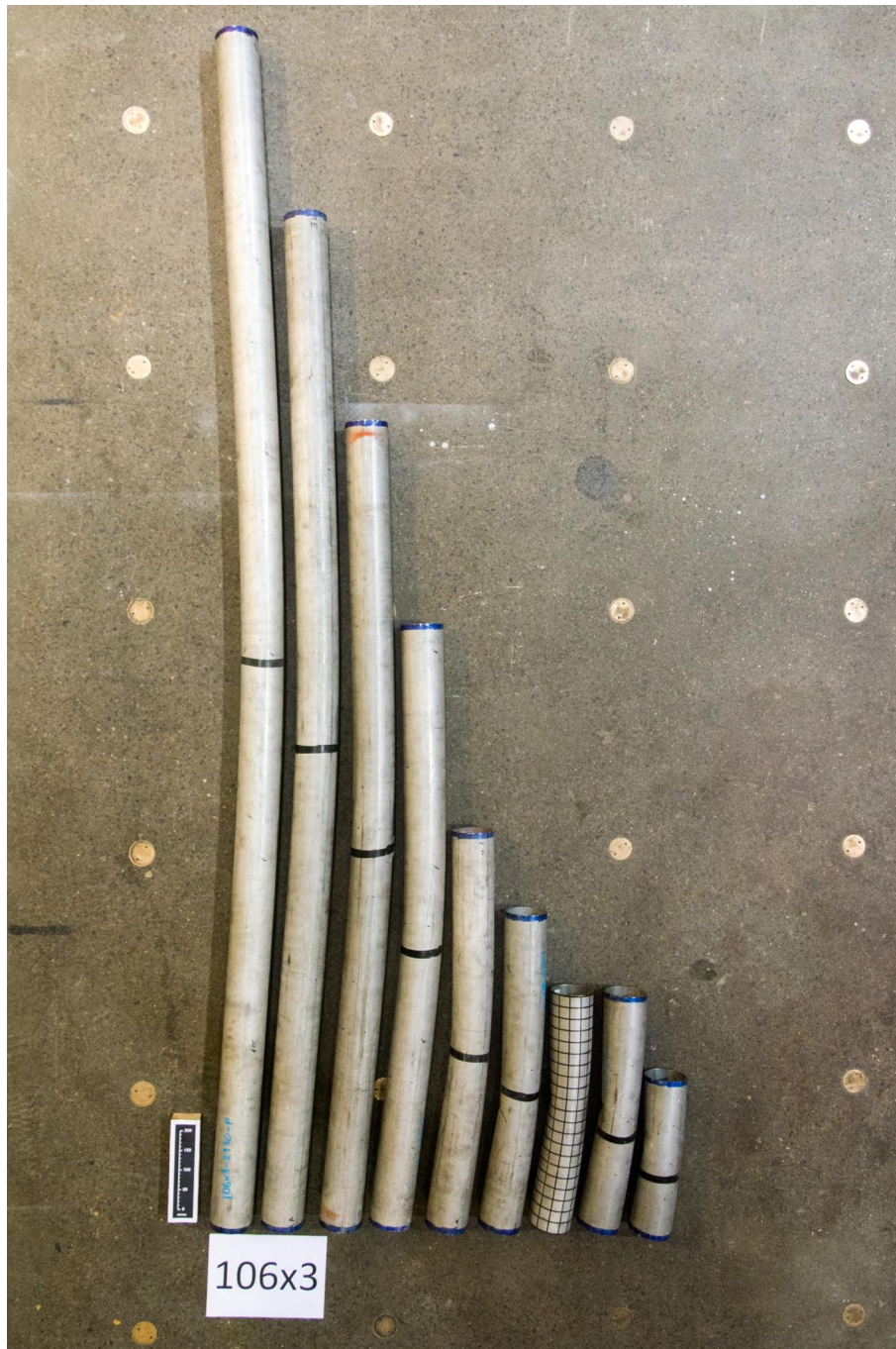


Figure 4.8: The deformed austenitic 106×3 CHS long columns.

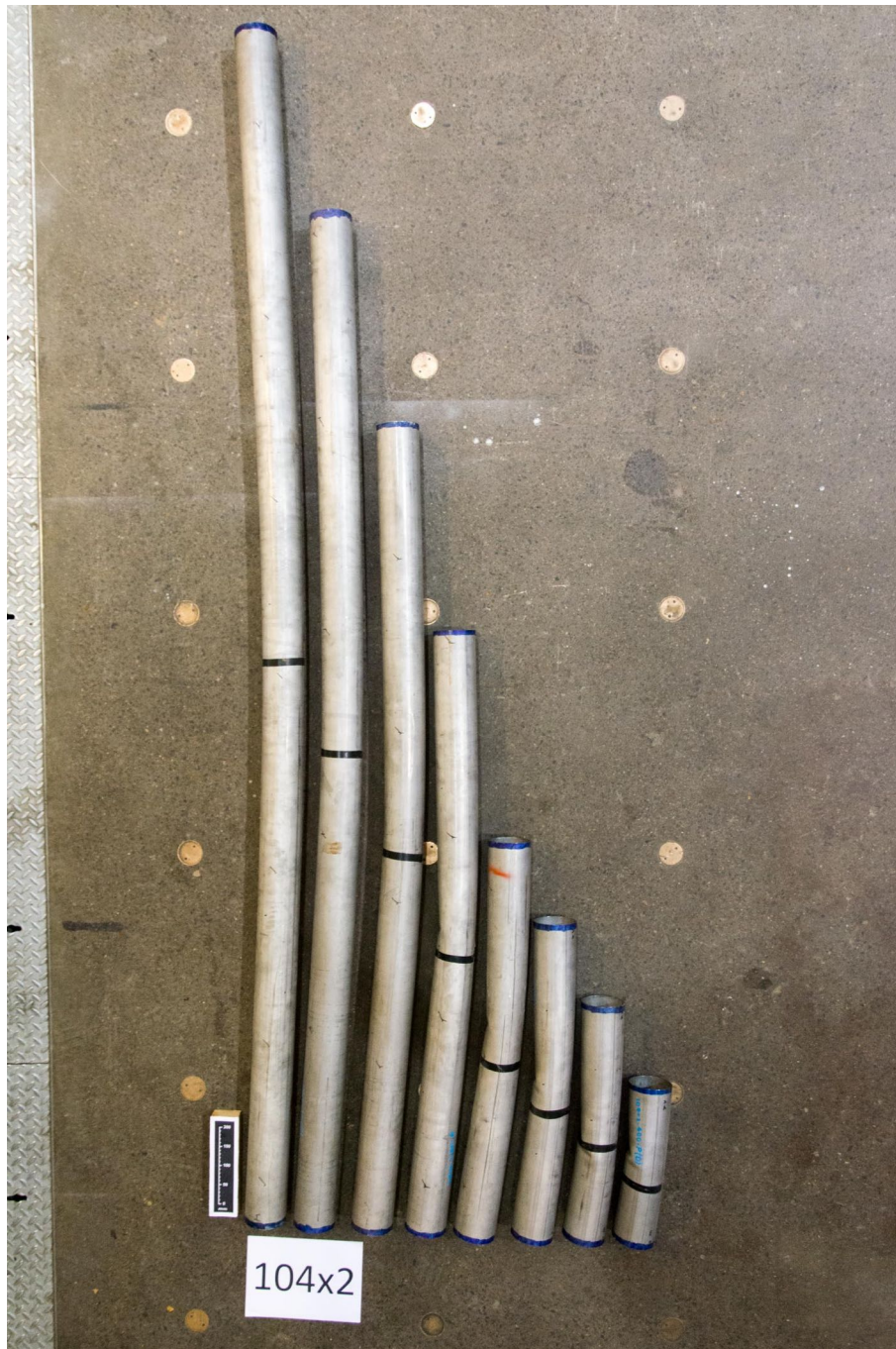


Figure 4.9: The deformed austenitic 104×2 CHS long columns.



Figure 4.10: The deformed duplex 88.9×2.6 CHS long columns.



Figure 4.11: The deformed ferritic 80×1.5 CHS long columns.



Figure 4.12: The deformed ferritic 101.6×1.5 CHS long columns.

carried out. Models of each of the experiments undertaken in Section 4.2 have been produced, with a validation study undertaken to determine the most appropriate material stress–strain relationship and imperfection amplitudes. The aim is to reproduce the full load–deformation relationships, ultimate loads and failures modes measured and observed in the experimental study. A parametric study was then undertaken varying the type of stainless steel, cross-section slenderness and global slenderness to produce comprehensive numerical data from which to evaluate existing flexural buckling curve proposals and to propose a new flexural buckling curve.

4.3.1 Geometry

The average measured outer diameter and wall thickness of their respective specimen were adopted in the finite element (FE) models. In Abaqus/CAE 2016 (Dassault Systèmes Simulia Corp., 2015), the circular cross-section was drawn, extruded to the measured average specimen length and then partitioned to allow a reference point to be placed centrally on the least compressed face at mid-height, allowing the mid-height deflection to be consistently outputted.

Zhao et al. (2015a), Zhao et al. (2015b) and Zhao et al. (2016c) employed symmetry in their CHS FE studies, modelling both half of the cross-section and half of the member length. This was initially attempted in this study although non-symmetric deformed local model shapes were observed in the symmetric models, which are subsequently used to simulate local imperfections, and it was decided to model the whole specimen. For some of the more locally slender FE models cross-section symmetry was used to halve the model size, due to the computing limitations of a desktop computer, with care taken to identify suitable imperfection modes.

4.3.2 Material model

The measured stress–strain material response was employed in the FE models in order to reproduce the experimental tests accurately. Tensile and compressive properties of stainless steel have been considered due to the non-symmetrical nature of the material stress–strain response (Gardner, 2005). Separate averaged tensile and compressive stress–strain relationships were produced by determining the compound Ramberg–Osgood material properties, reproduced in Section 2.2.7 as Equations 2.4 and 2.11 from Gardner and Ashraf (2006), for the individual tensile coupon and compressive stub column responses, calculating the respective averaged tensile and compressive parameters and then producing tensile and compressive stress–strain responses from the averaged parameters. The stub column compressive stress–strain relationships include the effects of local buckling and therefore at the $\sigma_{1.0}$ proof stress the tensile coupon stress–strain relationship was translated and utilised beyond this point, ensuring a continuous stress–strain relationship with an ultimate material stress σ_u .

Abaqus/CAE 2016 (Dassault Systèmes Simulia Corp., 2015) requires the material properties to be inputted as true stress and plastic strain (which account for the changing cross-sectional area during deformation), not the engineering stress and strain (utilising the original undeformed geometry) measured from the tensile coupon and stub column tests (Dassault Systèmes Simulia Corp., 2013). The true stress σ_{true} can be calculated using

$$\sigma_{\text{true}} = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}), \quad (4.2)$$

where σ_{nom} is the nominal engineering stress and ε_{nom} is the nominal engineering strain. The plastic strain $\varepsilon_{\text{plastic}}$ can be determined from

$$\varepsilon_{\text{plastic}} = \varepsilon_{\text{true}} - \frac{\sigma_{\text{true}}}{E} \quad (4.3)$$

and the true strain $\varepsilon_{\text{true}}$ can be calculated from

$$\varepsilon_{\text{true}} = \ln(1 + \varepsilon_{\text{nom}}). \quad (4.4)$$

As discussed in Section 2.4.1, the membrane residual stresses have been observed to be small and are neglected, while the through thickness residual stresses are considered to be present in the measured material stress–strain response and are therefore implicitly incorporated into the finite element models.

4.3.3 Boundary conditions

The boundary conditions in the numerical models replicate the boundary conditions in the flexural buckling experiments. The cylinder ends are constrained to the movement of a single reference point which is positioned at the rotation point of the knife edges (i.e. offset longitudinally from the end of the cylinder). The end boundary conditions are then applied to these reference points, with the bottom end of the column being restricted from moving in any direction and from rotating, apart from in the rotation axis of the knife edge, and the top end of the column is similarly restrained, except with longitudinal movement allowed for end shortening. The constraining to a reference point and boundary conditions ensure that the ends of the cylinder rotate about the knife edge while remaining flat and undistorted. The load is applied at the top reference point to simulate the load being applied from the testing machine through the knife edge.

Some of the class 4 models were not able to be run due to their size (the number of nodes and elements) and hence symmetry was utilised to reduce the model size. Half of the cross-section was drawn and then a symmetrical boundary condition was applied along the longitudinal line of symmetry that prevents physical translation across it and rotation perpendicular to its plane.

4.3.4 Mesh elements and size

Four-noded doubly curved S4R shell elements were utilised, as outlined in the literature review in Section 2.4.2 as these have been successfully used in a number of previous stainless steel hollow section finite element analysis studies.

A mesh validation study was undertaken on the austenitic stainless steel 106×3-1150-P specimen with the element size varying from $10t$ to $t/3$ and the ultimate load and mid-height deformations at the ultimate load compared. The variation of ultimate load with mesh size is shown in Figure 4.13, with the ultimate load normalised by the failure load from the finest mesh ($t/3$). The failure load can be observed to converge as the mesh size decreases, initially rapidly then large increases in the number of elements results in minimal accuracy gains. The variation of mid-height deflection, normalised by the deflection from the $t/3$ mesh, with mesh size is plotted in Figure 4.14, and it is apparent that this relationship is much more variable than for the ultimate load (which may be due to numerical instabilities). An element size with size $t \times t$ is adopted as it balances accuracy with computational efficiency, and this has also been adopted in previous studies of stainless steel CHS members (Zhao et al., 2015a,b, 2016c).

4.3.5 Geometric imperfections

Local and global geometric imperfections were incorporated into the finite element models as outlined in Section 2.4.3. Elastic buckling analyses were undertaken and the local and global imperfections were taken as the lowest positive local and global buckling modes respectively. The global imperfections are also used to model any applied loading eccentricity, i.e. the load is applied concentrically to the cylinder in the finite element models and any additional eccentricity required to attain the $L/1000$ imperfection is included in the global imperfection amplitude magnitude.

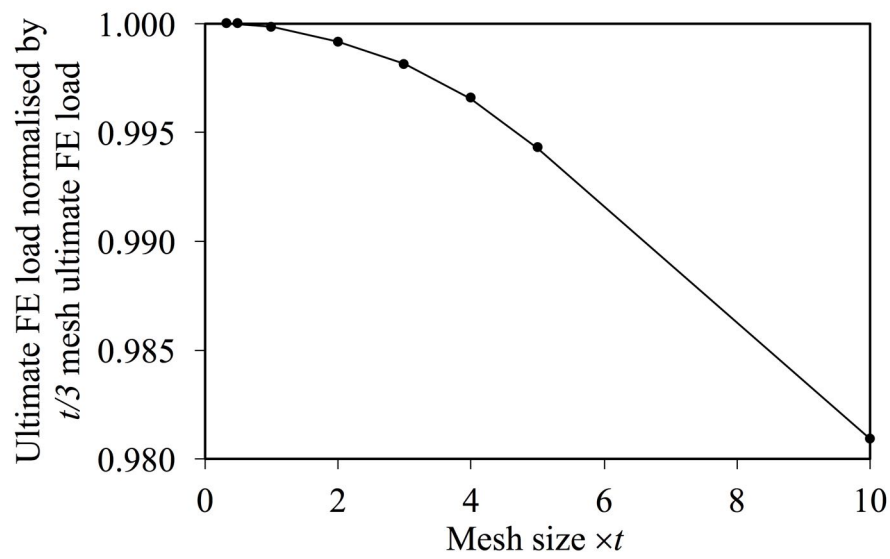


Figure 4.13: The variation of the normalised ultimate load with the mesh size for the 106 $\times$ 3-1150-P CHS specimen.

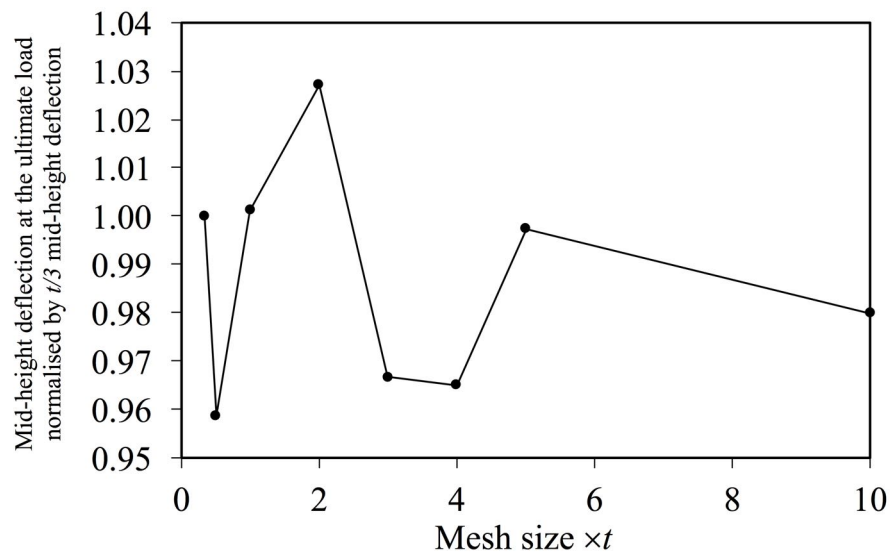


Figure 4.14: The variation of the normalised mid-height deflection with the mesh size for the 106 $\times$ 3-1150-P CHS specimen.

Zhao et al. (2015a) modelled austenitic stainless steel stub columns and eccentrically loaded stub columns and varied the amplitude of the local imperfection as $t/10$, $t/20$, $t/50$, $t/100$ and $t/500$ and observed that the peak load was relatively insensitive to the local imperfection amplitude, although the deformations at the ultimate load were more sensitive. The local amplitude of $t/100$ was determined to be the most appropriate in predicting the ultimate load and deformations. Zhao et al. (2016c) modelled concentrically and eccentrically loaded austenitic stainless steel columns. The local imperfection amplitudes modelled were $t/10$ and $t/100$, with global imperfection amplitudes of $L/1000$, $L/1500$ and the measured global imperfection ω_0 . It was noted that use of the measured global imperfections led to the least scattered predictions, although the best average agreement between the experimental and finite element values were for a local imperfection amplitude of $t/10$ and a global imperfection amplitude of $L/1000$.

In this study the local imperfection amplitudes modelled were $t/10$ and $t/100$, along with a global imperfection amplitude of the measured value ($\omega_0 + e_0$) and $L/1000$, and this is discussed further in Section 4.3.7. It is not always possible to achieve a global buckling mode for very short columns; if a global buckling mode was not attainable (in the first 200 modes) only a local buckling mode was used for the local imperfection.

4.3.6 Solver

The choice of the solver algorithms is outlined in the literature review in Section 2.4.4. The Lanczos method was used to obtain the buckling mode shapes, which were used as the local and global geometric imperfections, and the modified Riks method algorithm was used for the nonlinear geometric and material analysis.

4.3.7 Validation

Finite element models of the tested specimens were produced with the measured average geometric properties (D , t and L were different for each cylinder modelled), using both the average tensile properties from the tensile coupon tests and compressive properties

from the stub column tests and a local imperfection amplitude of $t/10$ and $t/100$ and a global imperfection amplitude of the measured value $(\omega_0 + e_0)$ and $L/1000$. The global imperfection is used to simulate both the initial global imperfection of the specimen and any additional applied eccentricity required to attain a combined equivalent global imperfection and applied eccentricity of $L/1000$. The finite element models are validated by comparing the predicted ultimate load and the deformation at the ultimate load with the experimental results. The load–deformation relationships and failure modes can also be compared between the physical experiments and numerical modelling.

The ultimate compressive capacity from the finite element model normalised by the measured experimental value $N_{u,FE}/N_{u,exp}$ are provided in Tables 4.6 to 4.11 for the individual specimen families and are averaged in Table 4.12. The ultimate load was generally well reproduced by the finite element models, with the models using the stub column material properties on average more accurate and less scattered than those using the tensile coupon material properties. The models are observed to be on average fairly insensitive to the local and global imperfection amplitudes, as highlighted by the models based on stub column properties having similar averages and identical coefficient of variation values (in Table 4.12); the most accurate predictions on average were obtained with a local imperfection amplitude of $t/10$ and using the measured global imperfection amplitude $(\omega_0 + e_0)$.

The mid-height deflection at the ultimate load from the finite element model normalised by the measured experimental value $\omega_{u,FE}/\omega_{u,exp}$ for the 88.9×2.6 and 80×1.5 CHS pin-ended columns are provided in Tables 4.13 and 4.14 respectively, and averaged for all columns in Table 4.15. The mid-height lateral deformation at the ultimate load is less successfully replicated by the finite element models than the ultimate load. Again the models are more accurate and consistent in their deformation predictions using the stub column material properties than the tensile coupon material properties. It can also be observed that on average the deformations are more sensitive to the imperfection

Table 4.6: $N_{u,FE}/N_{u,exp}$ for the 106×3 CHS pin-ended columns.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
106×3-400-F	0.99	0.99	1.01	1.01	1.01
106×3-550-P	1.10	1.10	1.14	1.14	1.14
106×3-750-P	0.99	0.99	1.08	1.08	1.08
106×3-950-P	0.88	0.88	0.95	0.95	0.95
106×3-1150-P	0.87	0.87	0.94	0.94	0.94
106×3-1650-P	0.94	0.94	1.00	1.00	1.00
106×3-2150-P	0.92	0.92	0.96	0.96	0.96
106×3-2650-P	0.98	0.98	0.99	0.99	0.99
106×3-3080-P	0.93	0.94	0.92	0.93	0.93
Mean $N_{u,FE}/N_{u,exp}$	0.956	0.956	0.999	0.999	0.999
COV $N_{u,FE}/N_{u,exp}$	0.073	0.073	0.069	0.069	0.069

Table 4.7: $N_{u,FE}/N_{u,exp}$ for the 104×2 CHS Finnish pin-ended columns.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
104×2-400-F	0.86	0.86	0.95	0.95	0.95
104×2-550-P	0.95	0.95	1.05	1.05	1.05
104×2-750-P	0.87	0.87	0.97	0.97	0.97
104×2-950-P	0.93	0.92	1.05	1.04	1.04
104×2-2650-P	0.98	0.98	1.02	1.02	1.02
Mean $N_{u,FE}/N_{u,exp}$	0.919	0.918	1.007	1.006	1.006
COV $N_{u,FE}/N_{u,exp}$	0.055	0.057	0.045	0.044	0.044

Table 4.8: $N_{u,FE}/N_{u,exp}$ for the 104×2 CHS Swedish pin-ended columns.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
104×2-400-FR	1.06	1.06	0.98	0.98	0.98
104×2-1150-P	1.09	1.09	1.04	1.05	1.05
104×2-1650-P	1.08	1.09	1.03	1.03	1.03
104×2-2150-P	1.10	1.08	1.03	1.01	1.01
104×2-3080-P	1.16	1.16	1.06	1.06	1.06
Mean $N_{u,FE}/N_{u,exp}$	1.099	1.095	1.028	1.025	1.025
COV $N_{u,FE}/N_{u,exp}$	0.036	0.038	0.029	0.032	0.031

Table 4.9: $N_{u,FE}/N_{u,exp}$ for the 88.9×2.6 CHS pin-ended columns.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
88.9×2.6-250-F	0.95	0.95	0.97	0.97	0.97
88.9×2.6-400-P	1.04	1.04	1.00	1.00	1.06
88.9×2.6-550-P	0.99	0.99	0.97	0.98	0.98
88.9×2.6-750-P	0.99	0.99	0.95	0.95	0.95
88.9×2.6-950-P	1.03	1.03	0.98	0.98	0.99
88.9×2.6-1150-P	1.11	1.12	1.05	1.05	1.05
88.9×2.6-1650-P	1.07	1.07	0.97	0.97	0.97
88.9×2.6-2150-P	1.18	1.13	1.07	1.04	1.04
88.9×2.6-2650-P	1.12	1.12	1.07	1.07	1.07
88.9×2.6-3080-P	1.08	1.08	1.07	1.07	1.07
Mean $N_{u,FE}/N_{u,exp}$	1.056	1.054	1.010	1.008	1.015
COV $N_{u,FE}/N_{u,exp}$	0.066	0.059	0.048	0.046	0.047

Table 4.10: $N_{u,FE}/N_{u,exp}$ for the 80×1.5 CHS pin-ended columns.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
80×1.5-350-F	0.99	0.99	0.98	0.98	0.98
80×1.5-300-P	1.00	1.00	0.98	0.98	0.98
80×1.5-450-P	1.05	1.05	1.04	1.04	1.04
80×1.5-700-P	1.04	1.04	1.02	1.03	1.03
80×1.5-900-P	1.03	1.03	1.01	1.01	1.01
80×1.5-1100-P	1.05	1.06	1.02	1.02	1.02
80×1.5-1600-P	1.12	1.12	1.03	1.03	1.03
Mean $N_{u,FE}/N_{u,exp}$	1.040	1.042	1.012	1.014	1.014
COV $N_{u,FE}/N_{u,exp}$	0.042	0.042	0.023	0.024	0.025

Table 4.11: $N_{u,FE}/N_{u,exp}$ for the 101×1.5 CHS pin-ended columns.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
101.6×1.5-400-F	1.02	1.02	0.98	0.98	0.98
101.6×1.5-350-P	1.03	1.03	0.98	0.98	0.98
101.6×1.5-500-P	1.05	1.05	0.99	0.99	1.00
101.6×1.5-800-P	1.11	1.11	1.06	1.06	1.06
101.6×1.5-1100-P	1.12	1.12	1.07	1.07	1.07
101.6×1.5-1600-P	1.18	1.19	1.07	1.08	1.08
Mean $N_{u,FE}/N_{u,exp}$	1.084	1.085	1.024	1.026	1.028
COV $N_{u,FE}/N_{u,exp}$	0.059	0.061	0.046	0.048	0.047

Table 4.12: $N_{u,FE}/N_{u,exp}$ summary.

Material model	Tensile properties		Stub properties		
	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
Mean $N_{u,FE}/N_{u,exp}$	1.025	1.024	1.012	1.011	1.013
COV $N_{u,FE}/N_{u,exp}$	0.082	0.081	0.046	0.046	0.046

Table 4.13: $\omega_{u,FE}/\omega_{u,exp}$ for the 88.9×2.6 CHS pin-ended columns.

Material model	Tensile properties		Stub properties		
	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
88.9×2.6-400-P	0.08	0.08	0.06	0.06	0.09
88.9×2.6-550-P	0.85	0.72	1.72	1.70	1.77
88.9×2.6-750-P	1.05	0.99	1.30	1.34	1.34
88.9×2.6-950-P	0.88	0.80	0.86	0.83	0.85
88.9×2.6-1150-P	0.68	0.73	0.82	0.81	0.79
88.9×2.6-1650-P	0.80	0.72	1.28	1.23	1.22
88.9×2.6-2150-P	0.68	0.79	0.79	0.91	0.87
88.9×2.6-2650-P	1.06	1.06	0.92	0.94	0.91
88.9×2.6-3080-P	1.41	1.38	1.08	1.02	1.06
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.832	0.809	0.982	0.982	0.990
COV $\omega_{u,FE}/\omega_{u,exp}$	0.433	0.432	0.465	0.457	0.462

amplitude than the ultimate load. The most accurate and least scattered deformation predictions are obtained using the stub column properties with a local imperfection amplitude of $t/10$ and a global imperfection amplitude of $L/1000$, though the results are very similar when the measured global imperfections are adopted.

The measured experimental and finite element analysis load mid-height lateral deformation curves for the 88.9×2.6-950-P and 106×3-1650-P CHS specimens are plotted in Figures 4.15 and 4.16 respectively, which show that in some cases the load–deformation history can be replicated numerically reasonably accurately. The failure modes of the 104×2-750-P and 106×3-3080-P CHS specimens for the experiments and finite element modelling are shown in Figures 4.17 and 4.18 and it is clear that the failure modes are well reproduced numerically.

Overall the most accurate predictions for ultimate load and mid-height deformation

Table 4.14: $\omega_{u,FE}/\omega_{u,exp}$ for the 80×1.5 CHS pin-ended columns.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
80×1.5 -300-P	0.84	0.84	0.97	0.97	0.61
80×1.5 -450-P	0.09	0.09	0.11	0.11	0.06
80×1.5 -700-P	0.62	0.61	0.58	0.51	0.51
80×1.5 -900-P	0.87	0.87	0.87	0.85	0.85
80×1.5 -1100-P	1.00	0.99	1.07	1.06	1.06
80×1.5 -1600-P	0.72	0.71	1.10	0.95	1.12
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.691	0.686	0.784	0.740	0.700
COV $\omega_{u,FE}/\omega_{u,exp}$	0.468	0.469	0.483	0.487	0.561

Table 4.15: $\omega_{u,FE}/\omega_{u,exp}$ summary.

Material model	Tensile properties		Stub properties		
Local imperfection amplitude	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection amplitude	$L/1000$	$\omega_0 + e_0$	$L/1000$	$\omega_0 + e_0$	$\omega_0 + e_0$
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.617	0.612	0.720	0.711	0.708
COV $\omega_{u,FE}/\omega_{u,exp}$	0.920	0.923	0.818	0.821	0.834

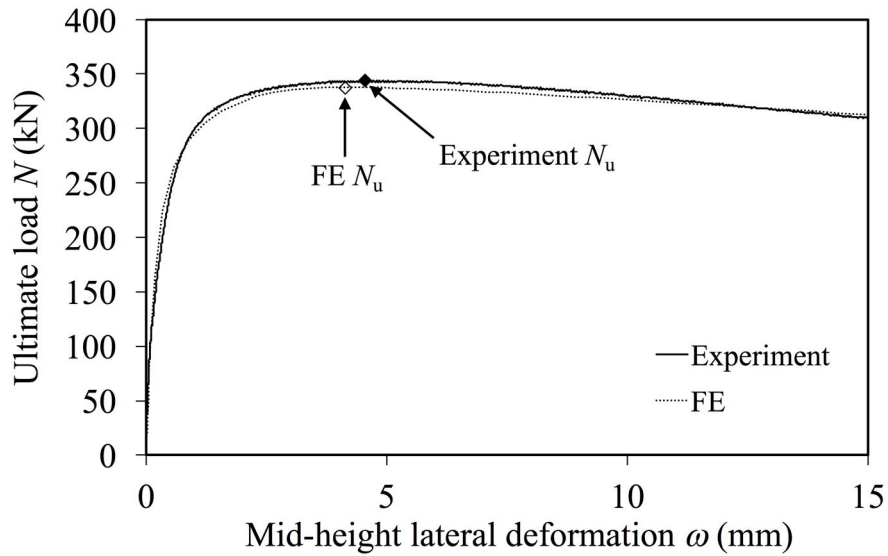


Figure 4.15: Experimental and FE load mid-height lateral deformation curves for the 88.9×2.6 -950-P CHS specimen.

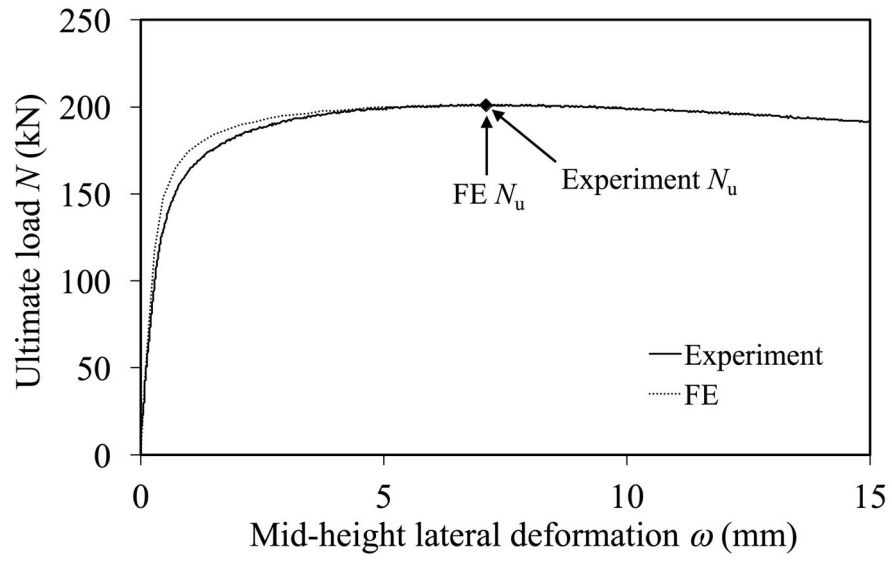


Figure 4.16: Experimental and FE load mid-height lateral deformation curves for the 106x3-1650-P CHS specimen.

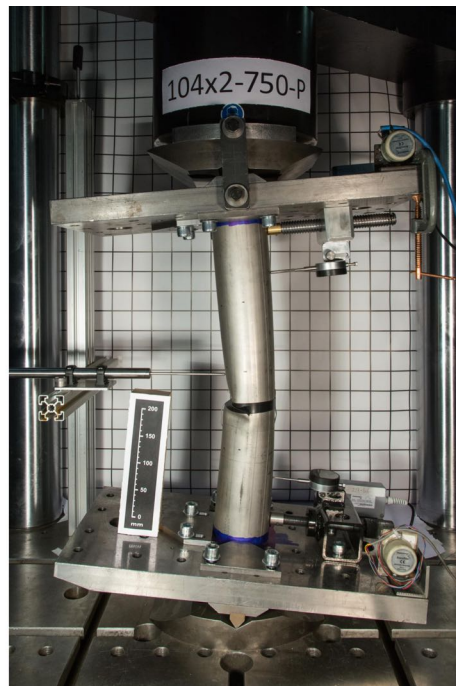


Figure 4.17: Experimental and FE failure modes for the 104x2-750-P CHS specimen.



Figure 4.18: Experimental and FE failure modes for the 106×3-3080-P CHS specimen.

are attained when using the stub column material properties with a local imperfection amplitude of $t/10$ and a global imperfection amplitude of the measured values or $L/1000$; it has also been shown that the load–deformation histories and failure modes are well replicated numerically.

4.3.8 Parametric study

A Python script was developed to allow a large number of finite element models to be produced quickly and efficiently. A representative cylinder was created in the Abaqus CAE environment for the elastic buckling analysis and nonlinear analysis and this was then turned into a script with the outer diameter D , wall thickness t , effective length L , Young’s modulus E and Poisson’s ratio ν as user definable variables, along with the measured stress–strain response. A 50 mm offset was applied between the cylinder end and the knife edge rotation point to simulate the same end plate and knife edge arrangement as in the UPC ferritic stainless steel specimen tests. The models used the stub column material properties, along with a $t/10$ local imperfection amplitude and

a $L/1000$ global imperfection. The final script allowed a range of cylinder lengths to be entered which were subsequently turned into individual input files. As discussed previously, in some cases global buckling modes were not attained from the elastic buckling analysis and only a local imperfection mode was used for these shorter column lengths. After the linear analysis, each model was inspected to determine the appropriate local and global buckling modes and these were then used in the nonlinear analyses. The nonlinear analyses were monitored and stopped after the peak load was reached to reduce unnecessary computation. Example Python scripts for the elastic buckling and the nonlinear analyses are provided in Appendix A.

Six cross-sections were modelled for each of the three common types of stainless steel used in construction. The austenitic cross-sections were based on the material properties from the 106×3 CHS cross-section, the duplex cross-sections from the 88.9×2.6 CHS cross-section and the ferritic cross-sections from the 101.6×1.5 CHS cross-section. The local slendernesses were varied over a wide range of values covering EN 1993-1-4 (2015) cross-section classes 1–4, with the member lengths chosen to provide a range of global slenderness values up to approximately $\bar{\lambda} = 2.5$. The modelled cross-sections are summarised in Table 4.16, where $D/(t\varepsilon^2)$ is the EN 1993-1-4 (2015) local slenderness, $\varepsilon = (235/f_y E/210000)^{0.5}$, $D/(t\varepsilon^2) \leq 50$ is class 1, $D/(t\varepsilon^2) \leq 70$ is class 2, $D/(t\varepsilon^2) \leq 90$ is class 3 and otherwise $D/(t\varepsilon^2) > 90$ is class 4, $\bar{\lambda}_{\min}$ and $\bar{\lambda}_{\max}$ are the minimum and maximum global slendernesses of the modelled cylinders and the number of FE models per cross-section is provided.

For some models computing limitations meant that reduced model sizes, exploiting symmetry, were required. The earlier Python scripts were modified to model only half of the cross-section with appropriate boundary conditions, as outlined previously in Section 4.3.3. The specimens where half the cross-section was modelled are noted in Table 4.16. The load–deformation histories for the 120×2.6 -1550-P and 65×1.5 -4400-P CHS models are plotted in Figures 4.19 and 4.20 respectively for the both the full and

Table 4.16: Summary of the concentrically loaded pin-ended parametric study specimens.

Section	Grade	Type	D (mm)	t (mm)	D/t	$D/(t\epsilon^2)$	EN 1993-1-4 (2015) compressive class	Local slenderness $\bar{\lambda}_c$	Global slenderness $\bar{\lambda}_{\min}$	$\bar{\lambda}_{\max}$	Number of FE models
50×3	1.4432	A	50	3.0	16.7	21.5	1	0.14	0.09	2.71	25
100×3	1.4432	A	100	3.0	33.3	43.1	1	0.20	0.05	2.70	26
150×3 ^a	1.4432	A	150	3.0	50.0	64.6	2	0.24	0.03	2.65	26
210×3 ^a	1.4432	A	210	3.0	70.0	90.5	4	0.29	0.04	2.73	26
270×3 ^a	1.4432	A	270	3.0	90.0	116.3	4	0.33	0.06	2.55	24
400×3	1.4432	A	400	3.0	133.3	172.3	4	0.40	0.05	2.62	21
25×2.6	1.4462	D	25	2.6	9.6	22.9	1	0.15	0.26	2.32	26
40×2.6	1.4462	D	40	2.6	15.4	36.6	1	0.18	0.15	2.33	23
55×2.6	1.4462	D	55	2.6	21.2	50.3	2	0.22	0.13	2.26	25
70×2.6	1.4462	D	70	2.6	26.9	64.0	2	0.24	0.10	2.25	25
120×2.6	1.4462	D	120	2.6	46.2	109.7	4	0.32	0.06	2.31	25
180×2.6 ^a	1.4462	D	180	2.6	69.2	164.5	4	0.39	0.03	2.37	27
25×1.5	1.4512	F	25	1.5	16.7	22.9	1	0.15	0.22	2.53	24
40×1.5	1.4512	F	40	1.5	26.7	36.6	1	0.18	0.14	2.52	27
65×1.5	1.4512	F	65	1.5	43.3	59.4	2	0.23	0.08	2.50	30
90×1.5	1.4512	F	90	1.5	60.0	82.3	3	0.28	0.06	2.48	28
150×1.5	1.4512	F	150	1.5	100.0	137.2	4	0.36	0.03	2.49	24
200×1.5	1.4512	F	200	1.5	133.3	182.9	4	0.41	0.02	2.54	27

^aHalf the cross-section was modelled

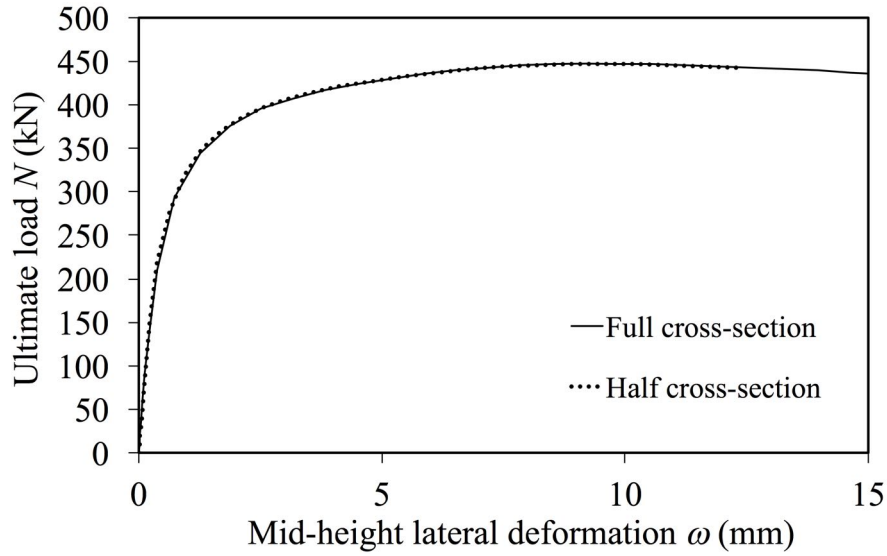


Figure 4.19: Load–deformation history for the 120×2.6-1550-P CHS model with a full and half cross-section modelled.

half cross-sections. The ultimate load from the full cross-section model normalised by the half cross-section value $N_{u,\text{full}}/N_{u,\text{half}}$ and the equivalent for the mid-height lateral deformation $\omega_{\text{full}}/\omega_{\text{half}}$ are provided in Table 4.17. It is apparent that the half cross-section models are essentially identical in the ultimate load, although for the mid-height deformation the half cross-sections models predict a slightly larger lateral deflection. The local and global modes for the full and half cross-sections are also listed in Table 4.17, the local and global modes are identical in shape and the mode number that they occur for the 120×2.6-1550-P CHS model, although they differ in terms of shape and the mode number at which the lowest local mode occurs for the 65×1.5-4400-P CHS model, with the local mode occurring at a much lower mode number.

4.4 Existing design methods

EN 1993-1-4 (2015) provides design rules for the flexural buckling resistance of stainless steel CHS columns. Other researchers have proposed improvements to these rules using both direct and iterative methods. Rasmussen and Rondal (2000), Young and Ellobody (2006), Ellobody and Young (2007) and Theofanous et al. (2009b) have all proposed changes to the existing buckling curve and their proposals are evaluated in the context of

Table 4.17: Comparison between models of full and half cross-section (with symmetrical boundary conditions).

Specimen	Grade	Type	D (mm)	t (mm)	L (mm)	Full cross-section		Half cross-section		$N_{u,full}/N_{u,half}$		$\omega_{full}/\omega_{half}$	
						Local mode	Global mode	Local mode	Global mode				
120×2.6-1550-P	1.4462	D	120	2.6	1550	2	1	2	1	1.00	1.00	0.96	0.96
65×1.5-4400-P	1.4512	F	65	1.5	4400	16	1	9	1	1.00	1.00	0.99	0.99

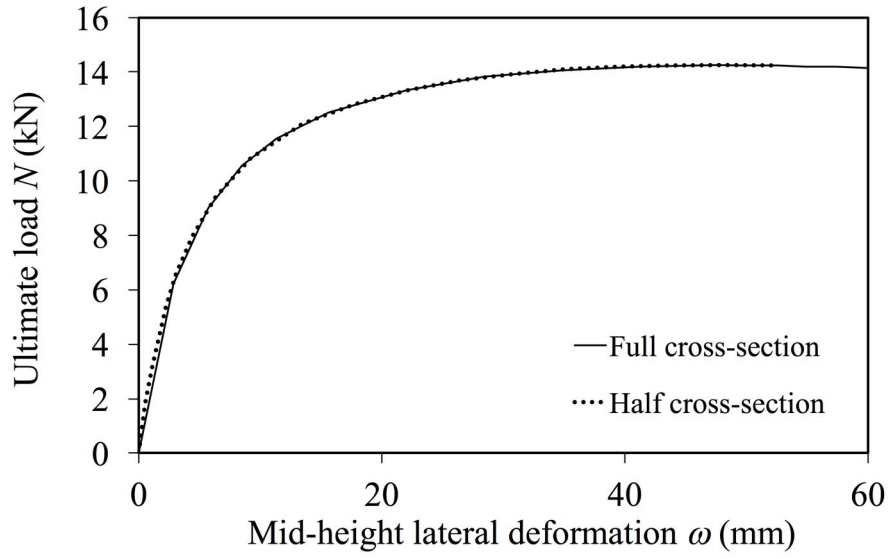


Figure 4.20: Load–deformation history for the 65×1.5-4400-P CHS model with a full and half cross-section modelled.

the collated existing experimental results and the new experimental and parametric FE results produced in this study. The measured geometric and material properties are used in the comparisons and all safety factors are taken to be equal to unity. The iterative methods proposed by Rasmussen and Hancock (1993a), Hradil et al. (2012) and Shu et al. (2014) have not been evaluated in this study.

4.4.1 EN 1993-1-4 (2015) design provisions

EN 1993-1-4 (2015) provides a single flexural buckling curve for welded and seamless stainless steel hollow sections. The flexural buckling resistance $N_{b,Rd}$ of members can be determined from

$$N_{b,Rd} = \frac{\chi A \sigma_{0.2}}{\gamma_{M1}} \text{ for class 1, 2 and 3 cross-sections} \quad (4.5)$$

and

$$N_{b,Rd} = \frac{\chi A_{eff} \sigma_{0.2}}{\gamma_{M1}} \text{ for class 4 cross-sections,} \quad (4.6)$$

where χ is the non-dimensional buckling reduction factor and γ_{M1} is the partial safety factor. The reduction factor χ is based upon the Perry curve (Ayrton and Perry, 1886) and can be determined from

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}}, \quad (4.7)$$

where ϕ is the operational parameter from

$$\phi = \frac{1}{2} \left(1 + \eta + \bar{\lambda}^2 \right). \quad (4.8)$$

The imperfection parameter η can be determined from

$$\eta = \alpha \left(\bar{\lambda} - \bar{\lambda}_0 \right), \quad (4.9)$$

where for stainless steel hollow cross-sections the imperfection factor $\alpha = 0.49$ and limiting slenderness $\bar{\lambda}_0 = 0.4$ (defining the length of the $\chi = 1$ plateau) which account for the effects of geometric imperfections and residual stresses.

The existing austenitic, duplex and ferritic stainless steel column test data have been added to that generated using Abaqus/CAE 2016 (Dassault Systèmes Simulia Corp., 2015) and are plotted with the reduction factor χ from EN 1993-1-4 (2015) in Figures 4.21 to 4.23. It is readily apparent from Figures 4.21 to 4.23 that the current EN 1993-1-4 (2015) design curve for stainless steel CHS columns is unsuitable for low and intermediate global slenderness values with unconservative predictions of the ultimate flexural buckling resistance. There are a number very unconservative resistance predictions for the fixed end column tests by Young and Hartono (2002), which form part of the existing austenitic column dataset in Figure 4.21; the normalised ultimate capacity of these columns is substantially lower than those obtained from other existing tests and the column tests undertaken in this study. In the very low slenderness range the data points for the stockier cross-sections are well in excess of the yield load due to strain hardening, which is not currently considered in EN 1993-1-4 (2015) (Buchanan

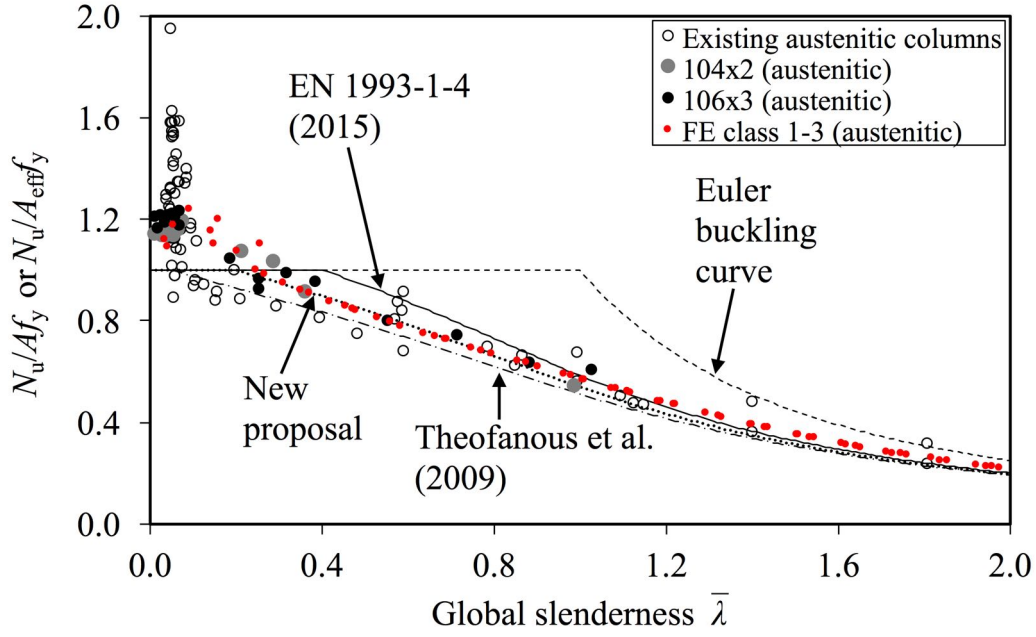


Figure 4.21: Comparison between the experimental and numerical results and single buckling curves for the austenitic CHS dataset.

et al., 2016a; Rasmussen and Rondal, 1997); this was addressed in Chapter 3.

4.4.2 Rasmussen and Rondal (2000) design provisions

Rasmussen and Rondal (2000) focused on producing flexural buckling curves for nonlinear materials and proposed a new nonlinear imperfection factor η expression,

$$\eta = \alpha \left((\bar{\lambda} - \lambda_1)^\beta - \lambda_0 \right). \quad (4.10)$$

The parameters α , β , λ_0 and λ_1 can be determined from the compressive Ramberg–Osgood material properties using Equations 4.11 to 4.14 (Rasmussen and Rondal, 1997), where $e = \sigma_{0.2}/E$.

$$\alpha = \frac{1.5}{(e^{0.6} + 0.03) (n^{(0.0048/e^{0.55})+1.4} + 13)} + \frac{0.002}{e^{0.6}} \quad (4.11)$$

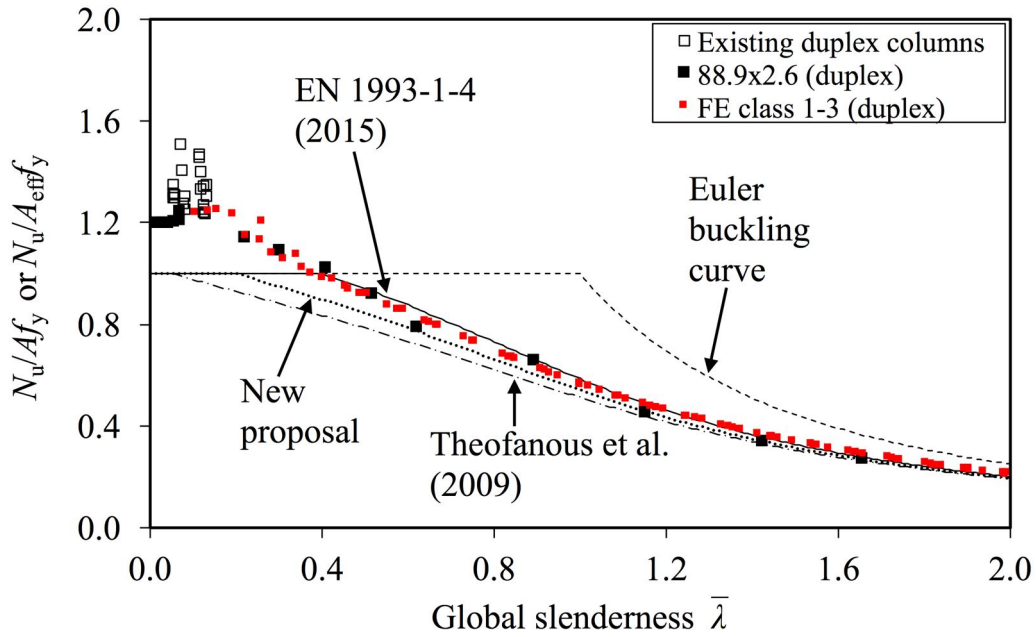


Figure 4.22: Comparison between the experimental and numerical results and single buckling curves for the duplex CHS dataset.

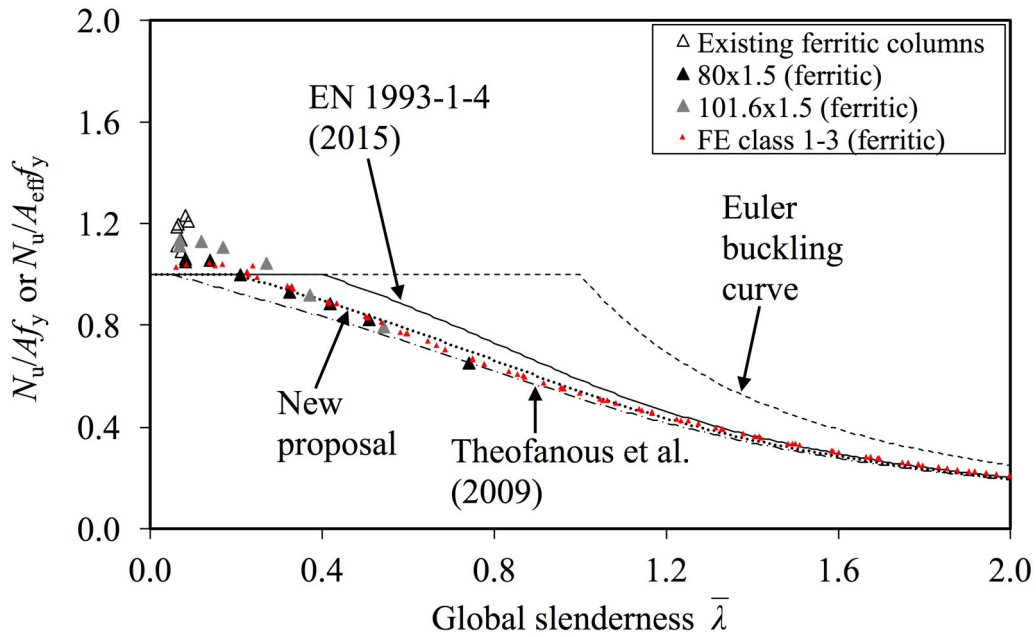


Figure 4.23: Comparison between the experimental and numerical results and single buckling curves for the ferritic CHS dataset.

$$\beta = \frac{0.36 \exp(-n)}{e^{0.45} + 0.007} + \tanh\left(\frac{n}{180} + \frac{6 \times 10^{-6}}{e^{1.4}} + 0.04\right) \quad (4.12)$$

$$\lambda_0 = 0.82 \left(\frac{e}{e + 0.0004} - 0.01n \right) \geq 0.2 \quad (4.13)$$

$$\lambda_1 = 0.8 \frac{e}{e + 0.0018} \left[1 - \left(\frac{n - 5.5}{n + (6e - 0.0054/e + 0.0015)} \right)^{1.2} \right] \quad (4.14)$$

Rasmussen and Rondal (2000) noted that due to the rounded nature of the material response of stainless steels the imperfection parameter η should have a nonlinear form and be a function of n and e to reflect the variation of the degree of roundness of the stress–strain curve with stainless steel grades.

4.4.3 Young and Ellobody (2006) design provisions

Young and Ellobody (2006) and Ellobody and Young (2007) undertook a parametric study on slender austenitic stainless steel fixed ended CHS columns and on austenitic and duplex non-slender fixed ended CHS columns. Two new buckling curves for CHS columns with non-slender cross-sections were proposed for normal strength (austenitic) and high strength (duplex) columns, with $\alpha = 0.8$ and $\bar{\lambda}_0 = 0.2$ and $\alpha = 0.4$ and $\bar{\lambda}_0 = 0.3$ in Equation 4.9 respectively. For slender cross-sections the existing EN 1993-1-4 (2015) column curve with a new effective area A_{eff} expression,

$$A_{\text{eff}} = A \varepsilon \left(\frac{125}{D/t} \right)^{0.1}, \quad (4.15)$$

was proposed. Ellobody and Young (2007) noted that the duplex stainless steel finite element results were reasonably well predicted by the existing EN 1993-1-4 (2015) buckling curve, which is generally also the case in this study as seen in Figure 4.22. The ferritic column results are taken to be normal strength stainless steel and the existing compressive EN 1993-1-4 (2015) class limits are applied in this study.

4.4.4 Theofanous et al. (2009b) design provisions

Theofanous et al. (2009b) proposed a new buckling curve for stainless steel CHS and EHS in the format of the existing EN 1993-1-4 (2015) expressions (as given by Equations 4.5 to 4.9), with an imperfection factor $\alpha = 0.49$ and limiting slenderness $\bar{\lambda}_0 = 0.05$ in Equation 4.9. The proposed buckling curve was established using experimental CHS and EHS stainless steel column buckling results and was statistically validated using EN 1990 (2005). This buckling curve has been plotted along with the experimental and numerical data in Figures 4.21 to 4.23. The same imperfection factor and limiting slenderness were also proposed by Ashraf et al. (2008a), although the 0.2% proof stress $\sigma_{0.2}$ was replaced with an effective buckling stress σ_{eff} as part of the early work on the continuous strength method, as outlined in the literature review in Section 2.3.2.

4.4.5 New flexural buckling curve proposal

A new flexural buckling curve is proposed that acts as a lower bound to the assembled experimental and numerical dataset, while still being efficient, that is not unwieldy for structural engineers to use and that achieves the target partial safety factor of $\gamma_{M1} = 1.1$ from EN 1993-1-4 (2015). The design curve is in the same format as the existing EN 1993-1-4 (2015) equations (as given by Equations 4.7 to 4.9), and has the same imperfection factor $\alpha = 0.49$, but with a reduced limiting slenderness $\bar{\lambda}_0 = 0.20$ in Equation 4.9. The new flexural buckling curve proposal is plotted in Figures 4.21 to 4.23 alongside the new and existing experimental and numerical data.

4.5 Discussion of existing and new design provisions

The highlighted existing and proposed design curves can be evaluated using the collated experimental and numerical column buckling dataset. The assembled dataset is plotted together with the existing EN 1993-1-4 (2015) buckling curve in Figures 4.21 to 4.23, along with the Euler buckling curve, the Theofanous et al. (2009b) proposal and the

new proposed flexural buckling curve. The other proposals cannot easily be compared in this form since the buckling curves vary with material properties. Comparisons between the ultimate compressive capacities and the predictions from the considered design methods for class 1–3 cross-sections are provided in Figures 4.24 to 4.26, which shows that all the considered proposals offer improved predictions of the ultimate capacity compared with EN 1993-1-4 (2015) for $\bar{\lambda} \leq 0.8$, although for more slender columns the Rasmussen and Rondal (2000) predictions are more unconservative than the current EN 1993-1-4 (2015) predictions. The average ratio of ultimate-to-predicted capacity is also reported in Table 4.18, along with coefficient of variation (COV) values, for class 1–3 cross-sections with the full range of global slendernesses $\bar{\lambda}$ and for columns with $\bar{\lambda} \leq 0.2$ and $\bar{\lambda} > 0.2$. The Rasmussen and Rondal (2000) predictions cannot be calculated for all of the collected literature data, as it requires compressive material properties which are rarely reported. This design method has also been applied using tensile material properties, although the strain hardening exponent n is not always provided in the literature and hence, where required, the proposals for n by Arrayago et al. (2015) have been used.

Overall it can be seen that the predictions with the Young and Ellobody (2006) and Ellobody and Young (2007) buckling curves are the most conservative of the design proposals considered. The proposals by Rasmussen and Rondal (2000) provide predictions that are generally the closest on average to the measured ultimate compressive capacities using both compressive and tensile material properties, although there are a large number of unconservative predictions. The compressive Rasmussen and Rondal (2000) predictions, with the smaller dataset against which to evaluate the proposal, are the most consistent with the lowest COV value. Theofanous et al. (2009b) and the new proposed flexural buckling curve predict intermediate resistances between Young and Ellobody (2006) and Ellobody and Young (2007), and Rasmussen and Rondal (2000). On average the new proposed curve provides estimates of the ultimate compressive capacity that are closer to that measured than Theofanous et al. (2009b).

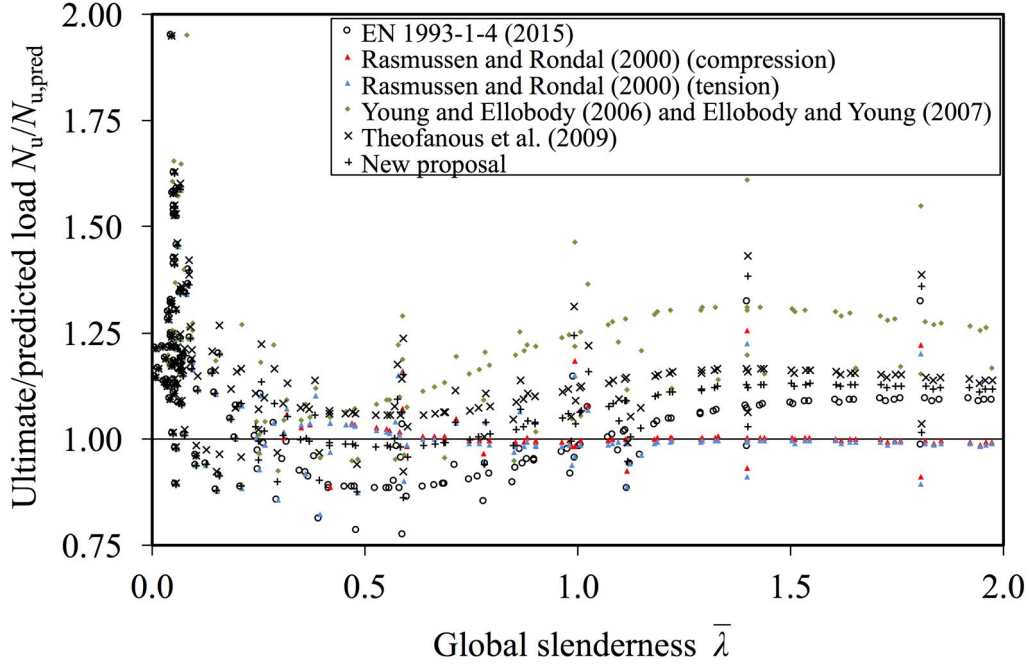


Figure 4.24: Comparisons of the existing design provisions with experimental and numerical data for austenitic class 1–3 cross-sections.

4.6 Reliability analyses

Design standards utilise partial factors on actions and resistance such that structures are designed with an appropriate level of reliability. In the European design standards the resistance prediction is divided by a partial factor γ_M (as seen in Section 4.4.1 in Equations 4.5 and 4.6), with γ_{M0} for the resistance at cross-section level to excessive yielding, γ_{M1} for member level resistance to instability and γ_{M2} for the resistance of cross-sections in tension to fracture and for connection resistances (bolts, rivets, welds, pin and plates). The partial factors are determined using a procedure outlined in Annex D of EN 1990 (2005); this incorporates the variation of the material and geometric properties, the resistance model and the number of test results used in the evaluation of the model (Baddoo, 2013; Afshan et al., 2015). At the present time $\gamma_{M0} = 1.1$, $\gamma_{M1} = 1.1$ and $\gamma_{M2} = 1.25$ in EN 1993-1-4 (2015).

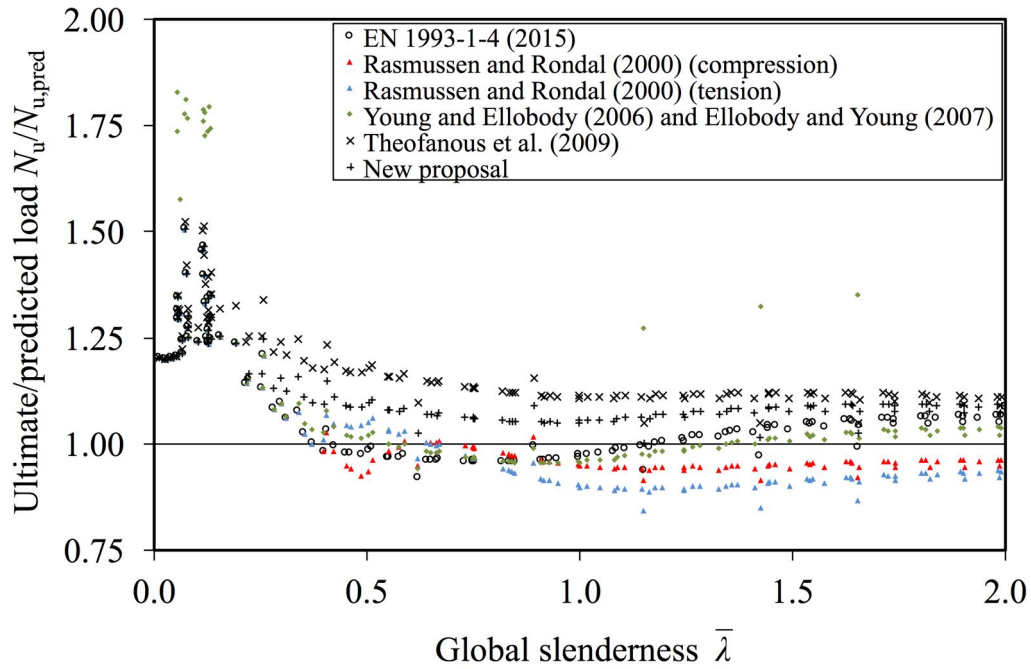


Figure 4.25: Comparisons of the existing design provisions with experimental and numerical data for duplex class 1–3 cross-sections.

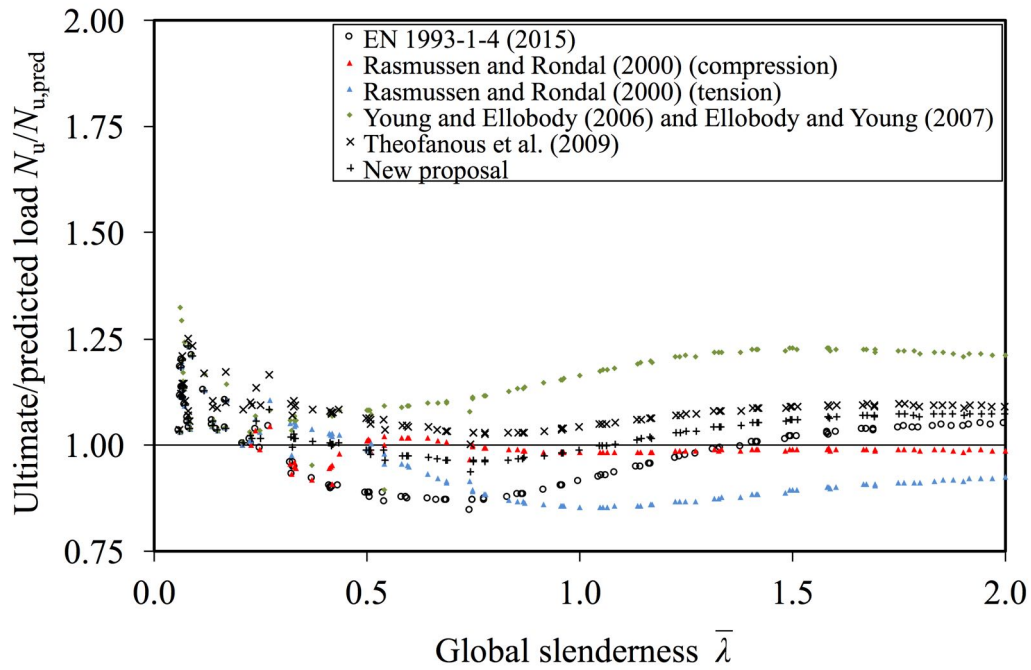


Figure 4.26: Comparisons of the existing design provisions with experimental and numerical data for ferritic class 1–3 cross-sections.

Table 4.18: Comparisons of the existing design capacity predictions with the experimental results for class 1–3 cross-sections.

Method	$N_u/N_{u,pred}$ (all)		$N_u/N_{u,pred}$ ($\bar{\lambda} \leq 0.2$)		$N_u/N_{u,pred}$ ($\bar{\lambda} > 0.2$)	
	Mean	COV	Mean	COV	Mean	COV
EN 1993-1-4 (2015)	1.06	0.14	1.22	0.14	1.00	0.08
Rasmussen and Rondal (2000) (compressive stub properties)	1.01	0.07	1.16	0.06	0.99	0.04
Rasmussen and Rondal (2000) (tensile coupon properties)	1.03	0.15	1.22	0.14	0.96	0.07
Young and Ellobody (2006) and Ellobody and Young (2007)	1.17	0.15	1.28	0.19	1.12	0.10
Theofanous et al. (2009b)	1.14	0.10	1.24	0.13	1.11	0.05
New proposal	1.10	0.11	1.22	0.14	1.06	0.06

The reliability analyses are undertaken for non-slender class 1–3 cross-sections only, as the slender class 4 predictions are relatively conservative due to the use of the effective cross-sectional area A_{eff} , and for columns where $\bar{\lambda} \geq \bar{\lambda}_0$, due to the lack of account taken for strain hardening for columns of low global slenderness leading to conservative predictions. The focus was therefore on cases where $\chi \leq 1$.

The current partial factors in EN 1993-1-4 (2015) are based on experiments undertaken in the 1990s and earlier, and it has been noted that since then, production techniques have improved and the stainless steel experimental dataset has significantly increased in size (Baddoo, 2013). Afshan et al. (2015) undertook an extensive re-evaluation of the EN 1993-1-4 (2015) partial factors and noted that the current CHS flexural buckling guidance is inappropriate for the adopted member level partial factor $\gamma_{M1} = 1.1$, although at the time only a limited dataset was available. The new experimental and numerical dataset produced in this study has been used to reassess the γ_{M1} partial factor with the current stainless steel hollow section buckling curve in EN 1993-1-4 (2015) as summarised in Table 4.19 (where n is the size of the dataset, the over-strength factor is the mean yield strength normalised by the nominal yield strength, V_{fy} is the coefficient of variation of the yield strength, V_{geometry} is the coefficient of variation of the geometry of the cross-section, $k_{d,n}$ is the fractile factor and is related to the size of the dataset, b is average ratio of the experimental to model resistance using a least squares fit and V_{δ} is the coefficient of variation of the tests relative to the resistance model). The over-strength factors, V_{fy} and V_{geometry} used in the reliability analysis were taken from Afshan et al. (2015). It is apparent that with a larger experimental and numerical dataset the γ_{M1} partial factor for the current EN 1993-1-4 (2015) flexural buckling curve should still be increased, although to a lower value than the analysis by Afshan et al. (2015) suggested.

The required γ_{M1} partial factor for the Theofanous et al. (2009b) and the new proposed buckling curve has been evaluated using the new experimental and numerical dataset with the results summarised in Tables 4.20 and 4.21 respectively. The new proposed

buckling curve has a lower average ratio of the experimental to model resistance b across the austenitic, duplex and ferritic datasets and is less scattered in terms of V_δ for the austenitic and duplex datasets, and slightly more scattered for the ferritic data, compared with the Theofanous et al. (2009b) buckling curve. The Theofanous et al. (2009b) buckling curve predictions typically have a higher V_δ compared with the EN 1993-1-4 (2015) flexural buckling curve. The γ_{M1} partial factors for the new proposal are closer to the current EN 1993-1-4 (2015) value of $\gamma_{M1} = 1.1$ than the current codified flexural buckling curve and less conservative than the Theofanous et al. (2009b) γ_{M1} partial factors for the duplex and ferritic datasets. The new proposed flexural buckling curve achieves approximately the required level of reliability in its flexural buckling capacity predictions.

4.7 Conclusions

A comprehensive experimental investigation focusing upon concentrically loaded austenitic, duplex and ferritic stainless steel circular hollow section columns has been undertaken. The programme consisted of material property tests, stub column tests and column member tests on non-slender class 1 to slender class 4 cross-sections. Additional data were produced using validated finite element methods and the experimental and numerical results have been compared against the current European design guidance and it was observed, similar to previous studies, that the current design approach can be unconservative. Existing non-iterative design proposals were then evaluated and were generally found to offer improved predictions of member level resistance. A new flexural buckling curve of the same form as the existing EN 1993-1-4 (2015) curve was proposed, with the same imperfection factor $\alpha = 0.49$ as the current code but with a reduced limiting slenderness $\bar{\lambda}_0 = 0.20$, with the aim of offering safe, but still efficient predictions, and a design approach that is not overly onerous to undertake. Reliability analyses were undertaken on the existing EN 1993-1-4 (2015) flexural buckling curve, it

Table 4.19: Summary of the statistical analysis results for the EN 1993-1-4 (2015) flexural buckling curve with $\alpha = 0.49$ and $\bar{\lambda}_0 = 0.4$.

Source	Type	Dataset	n	Over-strength	V_{fy}	V_{geometry}	$k_{d,n}$	b	V_{δ}	γ_{M1}
Afshan et al. (2015)	A	Previous tests	12	1.3	0.060	0.05	3.23	0.985	0.168	1.57
This study	A	Tests	23	1.3	0.060	0.05	3.11	0.965	0.133	1.43
This study	A	Tests and FE	89	1.3	0.060	0.05	3.11	0.954	0.098	1.33
This study	D	Tests	7	1.1	0.030	0.05	3.11	0.982	0.043	1.22
This study	D	Tests and FE	90	1.1	0.030	0.05	3.11	0.983	0.041	1.24
This study	F	Tests	3	1.2	0.045	0.05	3.11	0.877	0.030	1.24
This study	F	Tests and FE	95	1.2	0.045	0.05	3.11	0.905	0.075	1.37

Table 4.20: Summary of the statistical analysis results for the Theofanous et al. (2009b) flexural buckling curve with $\alpha = 0.49$ and $\bar{\lambda}_0 = 0.05$.

Type	Dataset	n	Over-strength	V_{fy}	V_{geometry}	$k_{d,n}$	b	V_{δ}	γ_{M1}
A	Tests	63	1.3	0.060	0.05	3.11	1.031	0.143	1.26
A	Tests and FE	139	1.3	0.060	0.05	3.11	1.043	0.099	1.12
D	Tests	20	1.1	0.030	0.05	3.11	1.242	0.094	1.05
D	Tests and FE	116	1.1	0.030	0.05	3.11	1.215	0.061	1.00
F	Tests	12	1.2	0.045	0.05	3.11	1.182	0.069	0.96
F	Tests and FE	117	1.2	0.045	0.05	3.11	1.154	0.032	0.93

Table 4.21: Summary of the statistical analysis results for the new proposed flexural buckling curve with $\alpha = 0.49$ and $\bar{\lambda}_0 = 0.2$.

Type	Dataset	n	Over-strength	V_{fy}	V_{geometry}	$k_{d,n}$	b	V_{δ}	γ_{M1}
A	Tests	30	1.3	0.060	0.05	3.11	1.017	0.115	1.25
A	Tests and FE	101	1.3	0.060	0.05	3.11	1.022	0.077	1.14
D	Tests	9	1.1	0.030	0.05	3.11	1.111	0.065	1.11
D	Tests and FE	101	1.1	0.030	0.05	3.11	1.099	0.027	1.06
F	Tests	5	1.2	0.045	0.05	3.11	0.988	0.029	1.09
F	Tests and FE	105	1.2	0.045	0.05	3.11	1.001	0.040	1.13

was seen that to achieve the required level of reliability in the member level resistance a partial safety factor larger than that currently utilised is required, although with the new proposed buckling curve the current partial safety factor is appropriate.

Chapter 5

CHS beam-columns

5.1 Introduction

Resistance predictions for stainless steel CHS under combined axial loading and bending moment at the cross-section level and for beam-columns at the member level are provided in EN 1993-1-4 (2015). It was apparent from the literature review in Chapter 2 that there is a lack of ferritic stainless steel CHS test data at both the cross-section and member level from which to appraise the existing design standard. The focus of this chapter is on the production of this new test data, additional finite element modelling, the evaluation of existing design guidance and the development of new design rules for stainless CHS beam-columns. The experimental programme has been published previously by Buchanan et al. (2016c).

5.2 Combined axial loading and bending moment and beam-column tests

5.2.1 General overview

The experimental investigation consisted of material property tests, stub column tests, cross-section level combined axial loading and bending moment tests and member level

beam-column tests on grade EN 1.4512 ferritic stainless steel CHS. Two different cross-section sizes were tested, with both sections close to the class 3 slenderness limit in compression — a class 3 80×1.5 CHS ($D/(t\epsilon^2) = 89.1$) and a class 4 101.6×1.5 CHS ($D/(t\epsilon^2) = 102.7$), according to the EN 1993-1-4 (2015) compressive class 3 limit of $D/(t\epsilon^2) = 90$. EN 1993-1-4 (2006) does not include guidance for the classification of CHS under combined loading, and hence linear interpolation between the compressive ($D/(t\epsilon^2) = 90$) and bending ($D/(t\epsilon^2) = 280$) class 3 limits was utilised, as discussed in Section 5.4.1. As a consequence, the eccentrically loaded specimens that were class 4 under pure compression became class 3 under combined bending and compression. The CHS were produced by cold-forming and laser welding. The tensile coupons were machined from the CHS and the material property tests carried out at Imperial College London (ICL), while the cross-section and member level tests were undertaken at Universitat Politècnica de Catalunya (UPC). The eccentrically loaded cross-section and member specimens were welded to 10 mm thick end plates to allow mechanical attachment to the knife edges. The specimen notation adopted described by means of the following example: the 80×1.5 -450-P-10E specimen is an 80×1.5 CHS cross-section with 450 mm effective length, 'P' pinned end conditions and '-10E' indicates an eccentrically loaded specimen with '10' being the nominal eccentricity in mm.

5.2.2 Material properties

Tensile coupon tests were undertaken to ascertain the material properties of the examined ferritic CHS cross-sections. These tests were described previously in Section 3.4.3.

5.2.3 Geometric properties

The geometric properties of the specimens were determined prior to testing. Measurements of the outer diameter were taken at three equally spaced longitudinal locations for the short pin-ended columns ($L \leq 500$ mm) and at five equally spaced longitudinal

locations for the longer beam-column specimens. At each longitudinal location the outer diameter was recorded in four evenly distributed orientations (at 45° intervals) with mechanical callipers, allowing the average outer diameter D of the specimen and the EN 10219-2 (2006a) 'out-of-roundness' O to be calculated using Equation 3.3 in Chapter 3. The maximum measured 'out-of-roundness' was $O = 1.9\%$ which is marginally less than the codified limit of $O \leq 2\%$. The average wall thickness t was determined by measuring the wall thickness at the ends of the tubes at eight equally spaced locations (at 45° intervals) around the circumference, at a distance not less than $2t$ from the weld location in accordance with EN 10219-2 (2006a). The wall thicknesses of the short combined loading and long beam-column specimens could not be measured prior to testing due to the welded end plates, however the thickness variation around the ferritic stub columns was small ($< 4.5\%$, < 0.06 mm) and the average wall thickness t for the ferritic stub columns was therefore used for the eccentrically loaded specimens. The average specimen length L , which for the pin-ended members includes the additional length from the end plates and knife edges, was calculated based on measurements taken at 90° equally spaced locations around the circumference of the CHS using a tape measure. The mid-point global imperfections ω_0 were recorded by measuring the deviation from a flat plane linking the two ends of the specimens, which was achieved using an aluminium extrusion that was placed over the end plates and the deviation measured using mechanical callipers. For the pin-ended specimens, the imperfection measurements were made on the two faces parallel to the knife-edge axis (i.e. to obtain the imperfection in the direction of buckling). The average geometric properties of the specimens are provided in Tables 5.1 and 5.2 for the short beam-columns and Tables 5.3 and 5.4 for the longer beam-columns. The position of the weld for the eccentric specimens was kept constant, with the weld orientated to be on the least compressed face.

5.2.4 Stub column tests

Stub column tests were carried out to ascertain the cross-sectional load carrying capacity under pure compression and to determine the compressive material properties. The stub column tests have been covered previously in Section 3.4.

5.2.5 Combined axial loading and bending moment test setup

Combined axial loading and bending moment tests on short members were undertaken to determine the cross-sectional resistance under combined loading. Applied load eccentricities varying from 5 mm to 130 mm were employed to produce a wide range of axial load to bending moment combinations. The length of the short beam-column specimens was the same as that of the stub columns, but with an additional 10 mm thick carbon steel end plate welded to each end. The end plates were then bolted to knife edges to create pin-ended conditions. An Instron 8805 testing machine was used for these tests, with the test setup shown in Figure 5.1. As with the previous ferritic stub column tests and shorter concentrically loaded column tests, described in Chapters 3 and 4 respectively, a spherical head was employed to ensure full contact between the specimen and the testing machine. Prior to testing the spherical head was loosened and checked for freedom of movement; the specimen was then loosely bolted in place and then an installation load of 5 kN was applied at which point the end plate and spherical head bolts were tightened, before removing the 5 kN installation load.

The end rotations were measured at both ends of the specimens using dual axis inclinometers, to monitor the rotations of the end plates parallel and perpendicular to the knife edges ($< 0.35^\circ$ perpendicular rotation was recorded and is attributed to the welded end plates not being perfectly aligned to the specimens). Two electrical resistance strain gauges were attached onto opposite faces at mid-height to measure the maximum and minimum strains from the combined compression and bending. The mid-height lateral deflection was monitored using a laser distance measurement sensor, positioned slightly above the strain gauge due to the need to reflect the laser back to the sensor

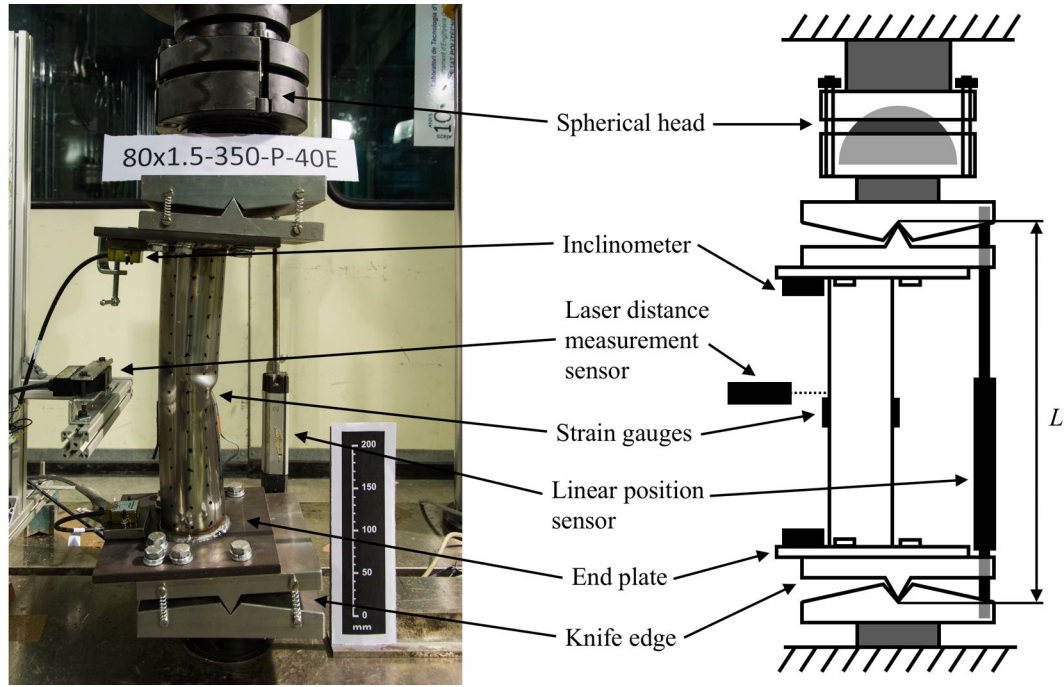


Figure 5.1: The short beam-column test setup.

(visible in Figure 4.4 from Chapter 4), and the applied load was again measured using a load cell incorporated within the testing machine. For end shortening, the position of the top platen was measured using an internal LVDT, inside the testing machine, along with an external linear position sensor attached between the outer non-rotating female ends of the knife edges. The parameters were recorded at a frequency of 2 Hz using a HBM MGCplus datalogger and Catman software. All of the tests were also filmed for later review and strain visualisation dots were painted onto the specimen surfaces as the reflective nature of the surface would make it difficult to observe small deformations without reference points.

The effective length of the specimens L is the distance between the knife edges, which were each 40 mm thick, as shown in Figure 5.1, resulting in global slenderness values of $\bar{\lambda} = 0.21$ for the 80×1.5 CHS and $\bar{\lambda} = 0.17$ for the 101.6×1.5 CHS, calculated using Equations 3.6 and 3.7 from Chapter 3. The top platen was moved at a displacement rate of 0.2 mm/min, in line with a previous study by Zhao et al. (2015a) on austenitic CHS under combined loading, and the tests were continued beyond the peak load until a knife edge was close to reaching its maximum rotation.

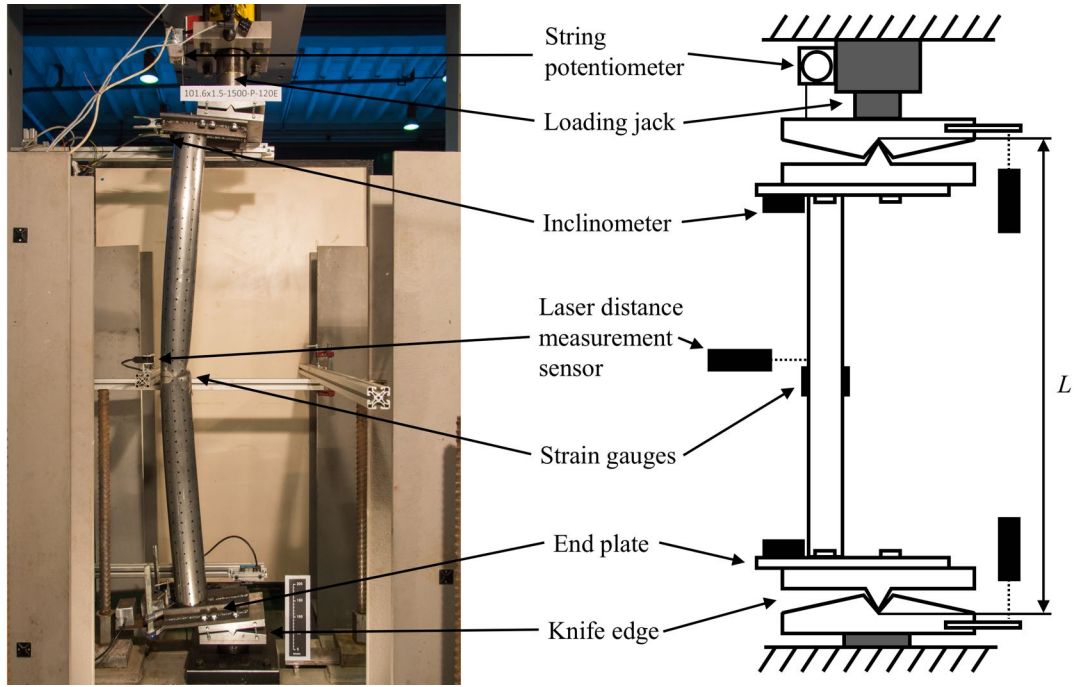


Figure 5.2: The long beam-column test setup.

5.2.6 Member beam-column test setup

Member level tests were carried out to examine the beam-column behaviour of ferritic stainless steel CHS columns. The specimens had a nominal length of 1600 mm, resulting in global slenderness values of $\bar{\lambda} = 0.74$ for the 80×1.5 CHS and $\bar{\lambda} = 0.56$ for the 101.6×1.5 CHS, calculated again from Equations 3.6 and 3.7 from Chapter 3. The test setup was similar to that used for the cross-section level tests in Section 5.2.5, an Ibertest MDA-700 testing machine was used to accommodate the longer test specimens, as shown in Figure 5.2. No spherical head was used for the long beam-columns due to limited headroom in the testing rig.

The instrumentation consisted of two dual axis inclinometers to record end rotations, two strain gauges affixed to the most and least compressed faces to measure the strains, a laser distance measurement sensor to monitor the mid-height lateral deflection, and pressure gauges to calculate the applied load. The end shortening of the specimens

was recorded using a string potentiometer, which was attached to and controlled the displacement of the loading jack, and secondly using two additional laser measurement devices monitoring the vertical displacement of the non-rotating female ends of the knife ends. The instrumentation parameters were again recorded at 2 Hz and the loading jack moved at a displacement rate of 0.2 mm/min.

5.2.7 Applied eccentricity

The loading line position was varied to obtain different ratios of axial load to bending moment. One concentrically loaded column for each cross-section was tested (reported previously in Chapter 4), with the loading eccentricity carefully set such that the sum of the applied eccentricity e_0 and initial global imperfection ω_0 was equal to $L/1000$ to simulate an imperfect column. As for the prior concentrically loaded columns, the applied eccentricities were determined by both physically measuring the distance from the centre of the specimen to the knife edge loading line and from back-calculating using the instrumentation readings during the initial elastic loading range. The physically measured eccentricities were recorded after the conclusion of each test prior to loosening of the end plate bolts. The calculated initial eccentricity e_0 can be determined from

$$e_0 = \frac{EI(\varepsilon_{\max} - \varepsilon_{\min})}{DN} - \omega - \omega_0, \quad (5.1)$$

using the Young's modulus E , second moment of area I , axial load N , average outer diameter D , mid-height lateral displacement ω and the maximum and minimum strains $\varepsilon_{\max}$ and $\varepsilon_{\min}$, which is a rearrangement of Equation 4.1 from Section 4.2.6. Overall the measured eccentricities were reasonably close to those calculated from the instrumentation, but the latter are taken to be more accurate (Huang and Young, 2014; Zhao et al., 2015c,e,a, 2016b) and therefore are used in the bending moment calculations.

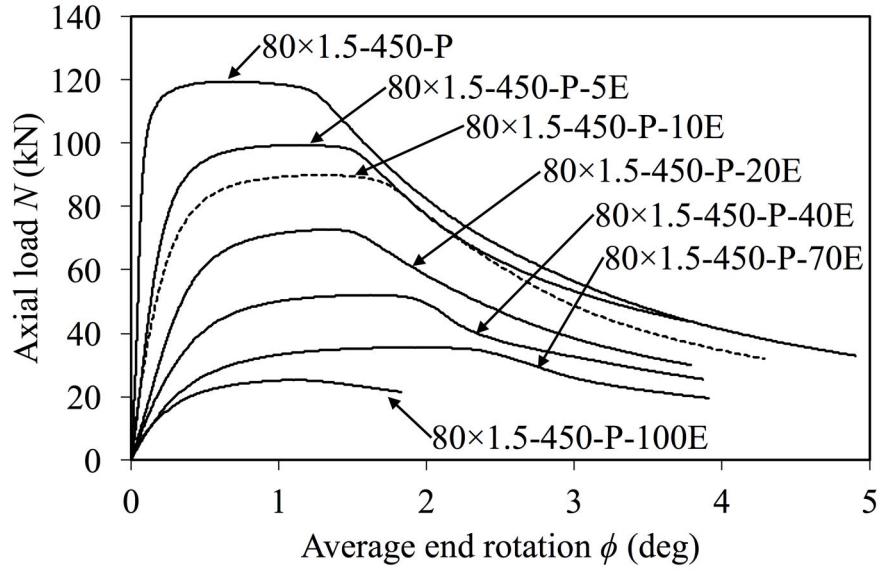


Figure 5.3: Load–end rotation plot for the 80×1.5 CHS short beam-column specimens.

5.2.8 Combined axial loading and bending moment test results

The average outer diameter D , wall thickness t , length L , initial global imperfection ω_0 , calculated load eccentricity e_0 , mid-height deflection at the ultimate load ω_u , ultimate axial load N_u , ultimate mid-height bending moment M_u and the average end rotation at the ultimate load ϕ_u are reported in Tables 5.1 and 5.2. The ultimate mid-height bending moment M_u is the second order inelastic bending moment determined using

$$M_u = N_u (e_0 + \omega_0 + \omega_u). \quad (5.2)$$

Axial load versus average end rotation curves are presented in Figures 5.3 and 5.4.

The deformed specimens are shown in Figure 5.5. Two distinct failure modes were observed, the classical ‘elephant’s foot’ local buckle close to one end of the specimen for the smaller eccentricities and a crumpling local buckle close to the mid-height of the specimen for the larger eccentricities. Poisson’s ratio effects were observed for the 101.6×1.5-500-P concentrically loaded CHS specimen, with the painted dots initially moving radially outwards before the specimen started deforming laterally, behaviour which has been previously observed in finite element modelling of stainless steel elliptical

Table 5.1: Summary of the geometric properties and test results for the short 80×1.5 CHS beam-column specimens.

Specimen	D (mm)	t (mm)	L (mm)	ω_0 (mm)	e_0 (mm)	ω_u (mm)	N_u (kN)	M_u (kNm)	ϕ_u (deg)
80×1.5 -450-P	80.01	1.34	448.9	0.29	0.11	1.71	119.4	0.25	0.66
80×1.5 -450-P-5E	80.03	1.34	449.1	0.18	5.79	2.49	99.4	0.84	1.10
80×1.5 -450-P-10E	79.98	1.34	449.4	0.37	8.47	3.02	89.9	1.07	1.33
80×1.5 -450-P-20E	79.99	1.34	449.4	0.31	21.63	2.86	72.6	1.80	1.33
80×1.5 -450-P-40E	79.99	1.34	449.0	0.22	38.37	3.27	52.0	2.18	1.60
80×1.5 -450-P-70E	80.01	1.34	449.1	0.16	71.08	3.75	35.5	2.66	1.89
80×1.5 -450-P-100E ^a	80.00	1.34	449.3	0.42	96.99	3.78	25.2	2.55	1.09

Table 5.2: Summary of the geometric properties and test results for the short 101.6×1.5 CHS beam-column specimens.

Specimen	D (mm)	t (mm)	L (mm)	ω_0 (mm)	e_0 (mm)	ω_u (mm)	N_u (kN)	M_u (kNm)	ϕ_u (deg)
101.6×1.5 -500-P	101.68	1.34	499.6	0.13	0.41	2.34	145.4	0.42	0.70
101.6×1.5 -500-P-10E	101.76	1.34	499.4	0.45	10.83	2.49	116.0	1.60	0.98
101.6×1.5 -500-P-15E	101.76	1.34	499.1	0.44	15.74	2.62	105.4	1.98	0.96
101.6×1.5 -500-P-30E	101.69	1.34	499.1	0.14	31.21	2.66	84.8	2.88	1.15
101.6×1.5 -500-P-60E	101.71	1.34	498.5	0.19	63.53	2.59	54.6	3.62	1.15
101.6×1.5 -500-P-90E	101.71	1.34	498.3	0.28	89.58	2.71	40.9	3.79	1.28
101.6×1.5 -500-P-130E	101.68	1.34	498.5	0.31	128.6	3.53	31.5	4.17	1.68

^aEnd plates yielded in bending during this test

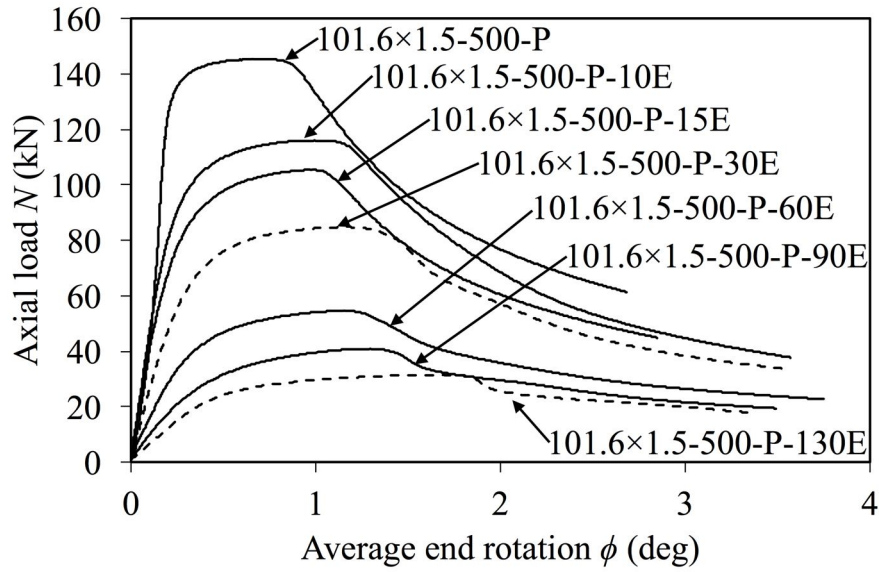


Figure 5.4: Load–end rotation plot for the 101.6×1.5 CHS short beam-column specimens.

hollow sections (EHS) by Theofanous et al. (2009b).

During the 80×1.5-450-P-100E CHS test it was noticed that the end plates were bending along the last line of bolts (visible in Figure 5.6), and after the test yield lines were clearly visible on both end plates. Consequently stiffening strips were welded along the end plates at both ends for the remaining short specimens and the untested beam-column specimens, as shown in Figure 5.7. In subsequent high eccentricity tests, a small amount of elastic deformation was visible beyond the last line of bolts, however there was no plastic deformation.

5.2.9 Member beam-column test results

The average geometric properties and key test results are reported in Tables 5.3 and 5.4, with the same notation as in Section 5.2.8. Axial load mid-height lateral deflection curves are presented in Figures 5.8 and 5.9. The failure mode of the specimens was global flexural buckling, followed by a mid-height local buckle during the post peak unloading for the 101.6×1.5 CHS specimens and all but the two most eccentrically loaded 80×1.5 CHS specimens. The deformed specimens are shown in Figures 5.10 and 5.11. The



Figure 5.5: The deformed short beam-columns, with load eccentricity increasing from left to right.

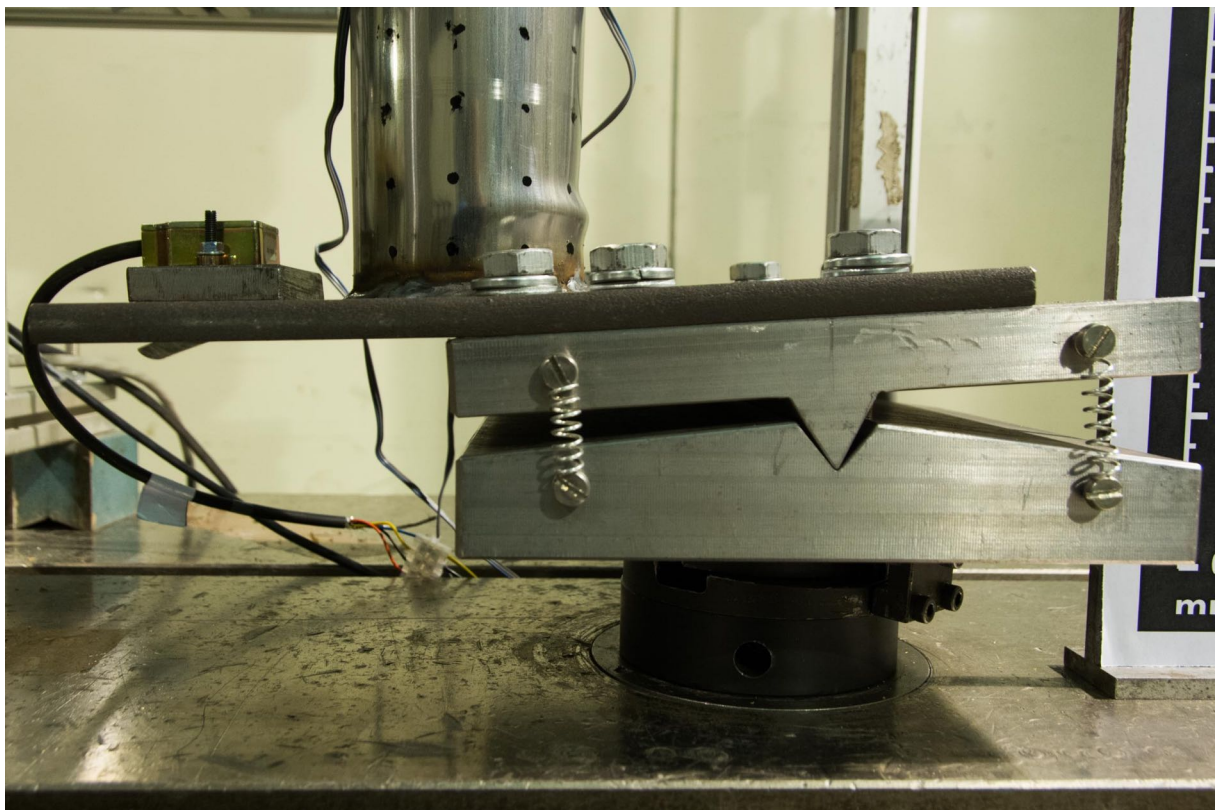


Figure 5.6: End plate bending about the last line of bolts for the 80×1.5-450-P-100E CHS specimen.



Figure 5.7: The deformed short beam-column with welded stiffeners on the end plates.

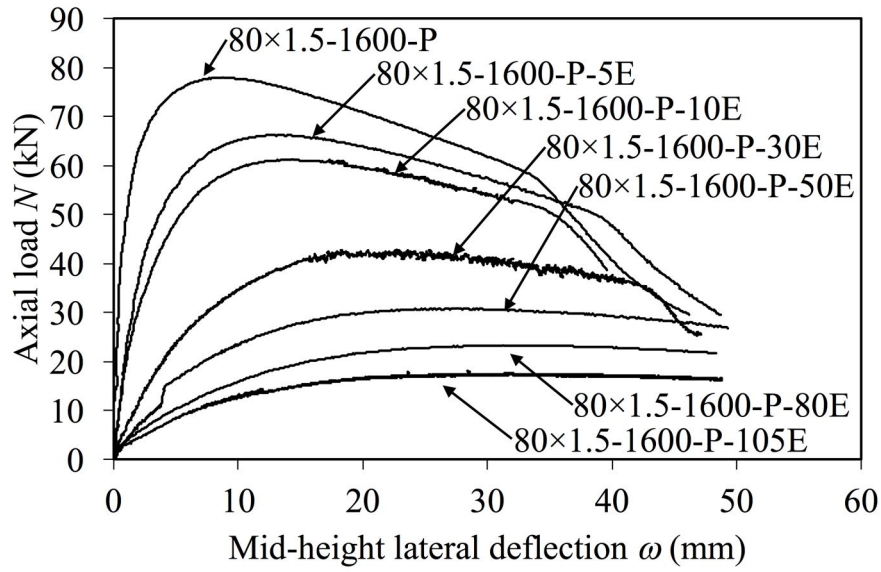


Figure 5.8: Load–mid height lateral deflection plot for the 80×1.5 CHS long beam-column specimens.

80×1.5-1600-P-30E CHS specimen was the first member tested, and hence the load mid-height lateral deflection curve has a greater degree of variation as the loading jack displacement control parameters were modified based on this test. The 101.6×1.5-1600-P-15E and 101.6×1.5-1600-P-25E CHS specimen tests were unsuccessful due to an issue with the string potentiometer which resulted in the control software not being able to monitor the position of the loading jack, and therefore applying increasing hydraulic pressure in an attempt to detect movement of the jack.

5.3 Finite element modelling

The finite element analysis package Abaqus/CAE 2016 (Dassault Systèmes Simulia Corp., 2015) has been used to undertake a series of geometrically and materially nonlinear analyses with imperfections (GMNIA) to produce additional stainless steel CHS beam-column data. The general approach outlined in Section 4.3 has been adopted, with any differences highlighted herein.

Table 5.3: Summary of the geometric properties and test results for the long 80×1.5 CHS beam-column specimens.

Specimen	D (mm)	t (mm)	L (mm)	ω_0 (mm)	e_0 (mm)	ω_u (mm)	N_u (kN)	M_u (kNm)	ϕ_u (deg)
80×1.5 -1600-P	80.00	1.34	1599.3	0.78	0.96	8.69	77.9	0.81	1.10
80×1.5 -1600-P-5E	80.01	1.34	1599.1	0.84	5.34	12.57	66.3	1.24	1.53
80×1.5 -1600-P-10E	80.06	1.34	1599.0	0.60	8.09	14.33	61.2	1.41	1.76
80×1.5 -1600-P-30E	80.01	1.34	1598.9	0.64	28.23	22.69	42.8	2.21	2.75
80×1.5 -1600-P-50E	80.03	1.34	1598.9	0.43	51.63	27.51	30.8	2.45	3.40
80×1.5 -1600-P-80E	80.04	1.34	1598.5	0.44	81.02	31.54	23.2	2.63	3.97
80×1.5 -1600-P-105E	80.01	1.34	1598.3	0.05	108.49	43.45	18.2	2.76	3.73

Table 5.4: Summary of the geometric properties and test results for the long 101.6×1.5 CHS beam-column specimens.

Specimen	D (mm)	t (mm)	L (mm)	ω_0 (mm)	e_0 (mm)	ω_u (mm)	N_u (kN)	M_u (kNm)	ϕ_u (deg)
101.6×1.5 -1600-P	101.71	1.34	1599.3	0.85	0.88	8.39	104.1	1.05	1.17
101.6×1.5 -1600-P-10E	101.67	1.34	1599.1	0.62	10.68	11.57	88.0	2.01	1.48
101.6×1.5 -1600-P-15E ^a	101.78	1.34	1599.0	0.91	-	-	-	-	-
101.6×1.5 -1600-P-25E ^a	101.73	1.34	1598.9	0.72	-	-	-	-	-
101.6×1.5 -1600-P-40E	101.73	1.34	1599.1	0.78	38.82	20.71	54.3	3.27	2.53
101.6×1.5 -1600-P-70E	101.73	1.34	1598.4	0.63	67.08	24.51	38.7	3.57	3.20
101.6×1.5 -1600-P-120E	101.72	1.34	1597.8	0.89	117.2	25.69	27.4	3.95	3.36

^aFailed tests due to loss of control of testing machine

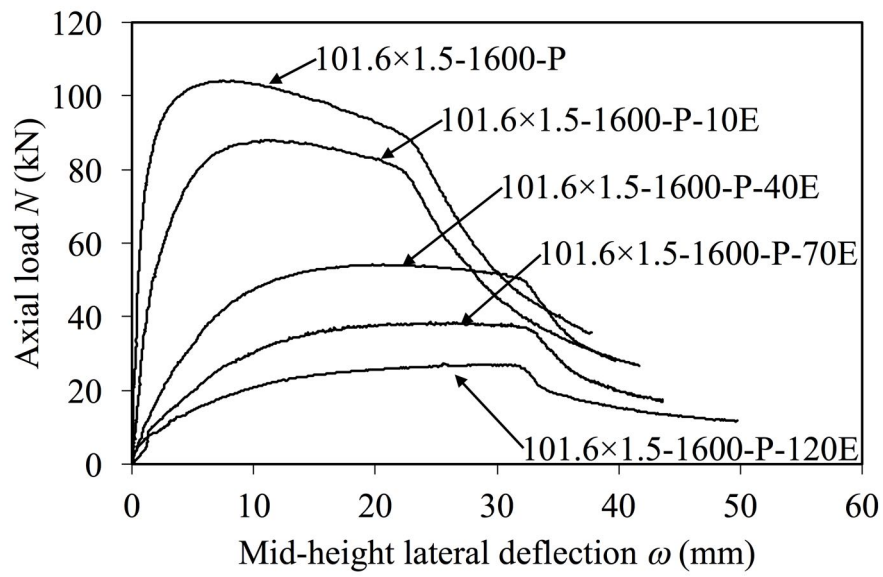


Figure 5.9: Load–mid height lateral deflection plot for the 101.6×1.5 CHS long beam-column specimens.



Figure 5.10: The deformed 80×1.5 CHS long beam-columns, with load eccentricity increasing from bottom to top.



Figure 5.11: The deformed 101.6×1.5 CHS long beam-columns, with load eccentricity increasing from bottom to top, including the unsuccessful test specimens.

5.3.1 Boundary conditions

The boundary conditions are identical to those used for the flexural buckling columns in Chapter 4, with the exception of the applied eccentricity. The reference points to which the column ends were constrained, the knife edges were positioned at and the load applied through were shifted laterally to simulate the applied eccentricity.

Symmetry was employed for the beam-column numerical models to decrease computational time, although as outlined in Chapter 4, care was taken to identify the appropriate local and global imperfection modes.

5.3.2 Validation

Numerical models of the tested specimens were created using the measured average geometric properties, with both the average tensile coupon properties and the compressive properties from the stub column tests. The cross-section level combined axial loading and bending moment models had a local imperfection amplitude of $t/10$ and $t/100$; the beam-column models also had a local imperfection amplitude of $t/10$ and $t/100$ and an additional global imperfection amplitude of the measured value ω_0 and $L/1000$. The applied eccentricity e_0 used was that calculated from Equation 5.1 in Section 5.2.7. The numerical models are validated by comparing the predicted ultimate load and the deformation at the ultimate load with the measured results.

The ultimate compressive capacities from the finite element model normalised by the measured experimental values $N_{u,FE}/N_{u,exp}$ are provided in Tables 5.5 and 5.6 for the cross-section level tests and in Tables 5.8 and 5.9 for the member level tests, with both averaged in Tables 5.7 and 5.10 respectively. The ultimate load is on average more closely predicted using the stub column material properties than the tensile coupon properties at both the cross-section and member level, with less scattered predictions at the member level. As with the previous concentrically loaded finite element models in Chapter 4, the analyses are relatively insensitive to the local and global imperfection amplitudes. The most accurate numerical predictions were on average attained with a local imperfection amplitude of $t/10$ and a global imperfection of $L/1000$.

The mid-height deflections at the ultimate load from the finite element model normalised by the measured experimental values $\omega_{u,FE}/\omega_{u,exp}$ are provided in Tables 5.11 and 5.12 for the cross-section level tests and in Tables 5.14 and 5.15 for the member level tests, with both averaged in Tables 5.13 and 5.16 respectively. Again, as seen in Chapter 4 with the concentrically loaded columns, the mid-height lateral deformation at the ultimate load is less successfully replicated by the finite element models than the ultimate load, particularly for the cross-section level models in Table 5.13. At the cross-section level the

Table 5.5: $N_{u,FE}/N_{u,exp}$ for the 80×1.5 CHS combined axial loading and bending moment columns.

Material model	Tensile properties		Stub properties	
Local imperfection	$t/10$	$t/100$	$t/10$	$t/100$
80×1.5 -450-P	1.05	1.05	1.04	1.03
80×1.5 -450-P-5E	1.04	1.04	1.01	1.02
80×1.5 -450-P-10E	1.07	1.08	1.05	1.05
80×1.5 -450-P-20E	1.00	1.01	0.98	0.98
80×1.5 -450-P-40E	1.05	1.06	1.03	1.03
80×1.5 -450-P-70E	1.02	1.03	0.98	1.00
80×1.5 -450-P-100E	1.13	1.13	1.09	1.10
Mean $N_{u,FE}/N_{u,exp}$	1.052	1.057	1.026	1.030
COV $N_{u,FE}/N_{u,exp}$	0.037	0.035	0.039	0.036

Table 5.6: $N_{u,FE}/N_{u,exp}$ for the 101.6×1.5 CHS combined axial loading and bending moment columns.

Material model	Tensile properties		Stub properties	
Local imperfection	$t/10$	$t/100$	$t/10$	$t/100$
101.6×1.5 -500-P	1.05	1.05	1.00	1.00
101.6×1.5 -500-P-10E	1.03	1.04	0.96	0.96
101.6×1.5 -500-P-15E	1.04	1.05	0.96	0.97
101.6×1.5 -500-P-30E	1.02	1.03	0.93	0.94
101.6×1.5 -500-P-60E	1.06	1.08	0.96	0.97
101.6×1.5 -500-P-90E	1.11	1.12	1.00	1.01
101.6×1.5 -500-P-130E	1.07	1.08	0.98	0.98
Mean $N_{u,FE}/N_{u,exp}$	1.056	1.064	0.971	0.975
COV $N_{u,FE}/N_{u,exp}$	0.028	0.030	0.025	0.025

Table 5.7: $N_{u,FE}/N_{u,exp}$ summary for the combined axial loading and bending moment columns.

Material model	Tensile properties		Stub properties	
Local imperfection	$t/10$	$t/100$	$t/10$	$t/100$
Mean $N_{u,FE}/N_{u,exp}$	1.054	1.061	0.998	1.003
COV $N_{u,FE}/N_{u,exp}$	0.032	0.032	0.043	0.041

Table 5.8: $N_{u,FE}/N_{u,exp}$ for the 80×1.5 CHS beam-columns.

Material model	Tensile properties		Stub properties		
Local imperfection	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection	L/1000	ω_0	L/1000	ω_0	ω_0
$80 \times 1.5-1600-P$	1.07	1.11	0.99	1.02	1.02
$80 \times 1.5-1600-P-5E$	1.08	1.11	1.02	1.03	1.03
$80 \times 1.5-1600-P-10E$	1.09	1.11	1.03	1.05	1.05
$80 \times 1.5-1600-P-30E$	1.06	1.07	1.00	1.01	1.01
$80 \times 1.5-1600-P-50E$	1.08	1.09	1.02	1.03	1.03
$80 \times 1.5-1600-P-80E$	1.07	1.08	1.02	1.03	1.03
$80 \times 1.5-1600-P-105E$	1.11	1.12	1.06	1.07	1.07
Mean $N_{u,FE}/N_{u,exp}$	1.080	1.098	1.019	1.035	1.035
COV $N_{u,FE}/N_{u,exp}$	0.016	0.017	0.022	0.019	0.019

Table 5.9: $N_{u,FE}/N_{u,exp}$ for the 101.6×1.5 CHS beam-columns.

Material model	Tensile properties		Stub properties		
Local imperfection	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection	L/1000	ω_0	L/1000	ω_0	ω_0
$101.6 \times 1.5-1600-P$	1.20	1.20	1.04	1.07	1.06
$101.6 \times 1.5-1600-P-10E$	1.07	1.09	0.98	1.00	1.00
$101.6 \times 1.5-1600-P-40E$	1.15	1.15	1.06	1.06	1.06
$101.6 \times 1.5-1600-P-70E$	1.18	1.19	1.10	1.10	1.10
$101.6 \times 1.5-1600-P-120E$	1.12	1.13	1.04	1.04	1.04
Mean $N_{u,FE}/N_{u,exp}$	1.144	1.151	1.043	1.052	1.052
COV $N_{u,FE}/N_{u,exp}$	0.044	0.039	0.040	0.035	0.035

Table 5.10: $N_{u,FE}/N_{u,exp}$ summary for the beam-columns.

Material model	Tensile properties		Stub properties		
Local imperfection	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection	L/1000	ω_0	L/1000	ω_0	ω_0
Mean $N_{u,FE}/N_{u,exp}$	1.107	1.120	1.029	1.042	1.042
COV $N_{u,FE}/N_{u,exp}$	0.042	0.036	0.032	0.027	0.027

Table 5.11: $\omega_{u,FE}/\omega_{u,exp}$ for the 80×1.5 CHS combined axial loading and bending moment columns.

Material model Local imperfection	Tensile properties		Stub properties	
	$t/10$	$t/100$	$t/10$	$t/100$
80×1.5 -450-P	0.00	0.04	-0.01	0.05
80×1.5 -450-P-5E	0.82	0.85	0.75	0.82
80×1.5 -450-P-10E	0.74	0.78	0.65	0.71
80×1.5 -450-P-20E	0.85	1.02	0.81	0.87
80×1.5 -450-P-40E	0.83	0.97	0.79	0.83
80×1.5 -450-P-70E	0.79	0.92	0.76	0.81
80×1.5 -450-P-100E	0.95	0.95	0.90	0.91
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.712	0.791	0.665	0.713
COV $\omega_{u,FE}/\omega_{u,exp}$	0.452	0.429	0.459	0.421

Table 5.12: $\omega_{u,FE}/\omega_{u,exp}$ for the 101.6×1.5 CHS combined axial loading and bending moment columns.

Material model Local imperfection	Tensile properties		Stub properties	
	$t/10$	$t/100$	$t/10$	$t/100$
101.6×1.5 -500-P	0.04	0.03	0.04	0.04
101.6×1.5 -500-P-10E	0.75	0.85	0.53	0.55
101.6×1.5 -500-P-15E	0.75	0.87	0.54	0.55
101.6×1.5 -500-P-30E	0.85	0.91	0.65	0.65
101.6×1.5 -500-P-60E	0.88	1.07	0.71	0.74
101.6×1.5 -500-P-90E	0.90	1.07	0.73	0.75
101.6×1.5 -500-P-130E	0.77	0.85	0.63	0.64
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.705	0.809	0.547	0.560
COV $\omega_{u,FE}/\omega_{u,exp}$	0.428	0.438	0.433	0.437

mid-height deflections are most accurately predicted using the tensile coupon material properties, while at the member level the stub column material properties provide the closest predictions. The mid-height deflections for the member level specimens are more accurate than for the cross-section specimens, and as with the ultimate load the most accurate numerical predictions were attained with a local imperfection amplitude of $t/10$ and a global imperfection of $L/1000$.

Overall the most accurate predictions for the ultimate load and mid-height deformation are generally achieved when using the stub column material properties with a local imperfection amplitude of $t/10$ and a global imperfection amplitude of $L/1000$ or the measured value.

Table 5.13: $\omega_{u,FE}/\omega_{u,exp}$ summary for the combined axial loading and bending moment columns.

Material model	Tensile properties		Stub properties	
Local imperfection	$t/10$	$t/100$	$t/10$	$t/100$
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.708	0.800	0.606	0.636
COV $\omega_{u,FE}/\omega_{u,exp}$	0.423	0.417	0.445	0.431

Table 5.14: $\omega_{u,FE}/\omega_{u,exp}$ for the 80×1.5 CHS beam-columns.

Material model	Tensile properties		Stub properties		
Local imperfection	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection	L/1000	ω_0	L/1000	ω_0	ω_0
80×1.5 -1600-P	0.84	0.73	1.29	1.18	1.18
80×1.5 -1600-P-5E	0.89	0.89	1.12	1.12	1.12
80×1.5 -1600-P-10E	0.92	0.89	1.07	1.06	1.05
80×1.5 -1600-P-30E	0.89	0.86	0.90	0.95	0.95
80×1.5 -1600-P-50E	0.84	0.83	0.91	0.92	0.92
80×1.5 -1600-P-80E	0.83	0.84	0.94	0.93	0.93
80×1.5 -1600-P-105E	0.66	0.65	0.76	0.76	0.76
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.838	0.813	0.997	0.989	0.987
COV $\omega_{u,FE}/\omega_{u,exp}$	0.103	0.110	0.173	0.146	0.145

Table 5.15: $\omega_{u,FE}/\omega_{u,exp}$ for the 101.6×1.5 CHS beam-columns.

Material model	Tensile properties		Stub properties		
Local imperfection	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection	L/1000	ω_0	L/1000	ω_0	ω_0
101.6×1.5 -1600-P	0.53	0.51	1.03	1.00	1.04
101.6×1.5 -1600-P-10E	0.85	0.82	1.14	1.12	1.12
101.6×1.5 -1600-P-40E	0.76	0.76	0.86	0.86	0.86
101.6×1.5 -1600-P-70E	0.74	0.76	0.85	0.85	0.85
101.6×1.5 -1600-P-120E	0.83	0.83	0.94	0.96	0.95
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.739	0.735	0.964	0.957	0.964
COV $\omega_{u,FE}/\omega_{u,exp}$	0.172	0.175	0.126	0.115	0.120

Table 5.16: $\omega_{u,FE}/\omega_{u,exp}$ summary for the beam-columns.

Material model	Tensile properties		Stub properties		
Local imperfection	$t/10$	$t/10$	$t/10$	$t/10$	$t/100$
Global imperfection	L/1000	ω_0	L/1000	ω_0	ω_0
Mean $\omega_{u,FE}/\omega_{u,exp}$	0.797	0.780	0.983	0.976	0.977
COV $\omega_{u,FE}/\omega_{u,exp}$	0.140	0.140	0.151	0.129	0.130

5.3.3 Parametric study

Zhao et al. (2015b) undertook an extensive finite element study on stainless steel CHS under combined axial loading and bending moment at the cross-sectional level, and therefore in this parametric study only stainless steel CHS beam-column members were modelled. A Python script was again used to allow a large number of finite element models to be produced and run quickly and efficiently. The development work was outlined in Section 4.3.8 and example code is provided in Appendix A. In total, 25 cross-sections were modelled: six austenitic cross-sections were assigned the material properties from the 106×3 CHS cross-section, nine duplex cross-sections from the 88.9×2.6 CHS cross-section and ten ferritic cross-sections from the 101.6×1.5 CHS cross-section. The local slendernesses were varied to model EN 1993-1-4 (2015) cross-section classes 1–3, the member lengths were chosen to provide a range of global slenderness values up to a maximum of $\bar{\lambda} = 2.6$ and the eccentricities varied from 0 mm to 805 mm to provide a range of axial load to bending moment ratios. The modelled cross-sections are summarised in Table 5.17, where $D/(t\varepsilon^2)$ is the EN 1993-1-4 (2015) local slenderness, $\varepsilon = (235/f_y E/210000)^{0.5}$, $\bar{\lambda}_{\min}$ and $\bar{\lambda}_{\max}$ are the minimum and maximum global slendernesses and $e_{0,\min}$ and $e_{0,\max}$ are the minimum and maximum applied eccentricities of the modelled cylinders, and the number of FE models per cross-section is provided.

5.4 Comparisons with existing design provisions

EN 1993-1-4 (2015) includes guidance for the design of both cross-sections and members under combined axial loading and bending moment for stainless steel CHS. The continuous strength method (CSM) has also been developed for stainless steel CHS cross-sections under combined axial loading and bending moment. The suitability of the current and proposed design guidance is assessed in this section using the experimental results at the cross-section level and the new and existing member level test results and

Table 5.17: Summary of the eccentricity loaded pin-ended parametric study specimens.

Section	Grade	Type	D (mm)	t (mm)	D/t	$D/(t\epsilon^2)$	EN 1993-1-4 (2015) compressive class	Local slenderness $\bar{\lambda}_c$	Global slenderness		Eccentricity (mm)		Number of FE models
									$\bar{\lambda}_{\min}$	$\bar{\lambda}_{\max}$	$e_{0,\min}$	$e_{0,\max}$	
100×8	1.4432	A	100	8.0	12.5	17.8	1	0.13	0.39	1.83	0	450	81
100×6	1.4432	A	100	6.0	16.7	23.7	1	0.15	0.25	2.59	7	750	96
100×5	1.4432	A	100	5.0	20.0	28.4	1	0.16	0.42	1.51	0	190	68
100×4	1.4432	A	100	4.0	25.0	35.5	1	0.18	0.56	2.05	0	550	94
100×3	1.4432	A	100	3.0	33.3	47.4	1	0.21	0.26	2.59	7	750	140
100×2	1.4432	A	100	2.0	50.0	71.0	3	0.26	0.44	1.90	0	350	115
100×11	1.4462	D	100	11.0	9.1	25.4	1	0.15	0.43	2.59	10	780	94
100×9	1.4462	D	100	9.0	11.1	31.0	1	0.17	0.83	1.93	0	420	53
100×8	1.4462	D	100	8.0	12.5	34.9	1	0.18	0.65	1.69	5	400	55
100×7	1.4462	D	100	7.0	14.3	39.9	1	0.19	0.54	1.62	0	500	56
100×5.5	1.4462	D	100	5.5	18.2	50.8	2	0.22	0.43	1.49	0	500	94
100×5	1.4462	D	100	5.0	20.0	55.9	2	0.23	0.63	1.06	5	140	22
100×4.5	1.4462	D	100	4.5	22.2	62.1	2	0.24	0.89	2.37	1	320	80
100×4	1.4462	D	100	4.0	25.0	69.8	2	0.25	0.42	0.94	0	420	35
100×3.5	1.4462	D	100	3.5	28.6	79.8	3	0.27	0.43	2.59	0	805	132
100×8	1.4512	F	100	8.0	12.5	20.3	1	0.14	0.58	2.04	0	260	55
100×6	1.4512	F	100	6.0	16.7	27.1	1	0.16	0.24	2.59	9	760	116
100×5	1.4512	F	100	5.0	20.0	32.5	1	0.17	0.52	1.78	0	500	95
100×4	1.4512	F	100	4.0	25.0	40.7	1	0.19	0.32	1.48	0	550	97
100×3.5	1.4512	F	100	3.5	28.6	46.5	1	0.21	1.69	1.98	380	590	16
100×3	1.4512	F	100	3.0	33.3	54.2	2	0.22	0.40	1.58	0	550	75
100×2.6	1.4512	F	100	2.6	38.5	62.6	2	0.24	0.71	1.18	2	320	39
100×2.3	1.4512	F	100	2.3	43.5	70.7	3	0.26	0.30	1.41	0	320	70
100×2.2	1.4512	F	100	2.2	45.5	73.9	3	0.26	0.28	1.41	0	230	108
100×2	1.4512	F	100	2.0	50.0	81.3	3	0.27	0.24	2.59	9	720	130

numerical models at the member level. The measured geometric and material properties are used in the comparisons and all safety factors are taken to be equal to unity.

5.4.1 EN 1993-1-4 (2015) design provisions for the cross-section behaviour

The current EN 1993-1-4 (2015) guidance for CHS under combined axial loading and bending moment adopts the same approach as for structural carbon steel in EN 1993-1-1 (2014). A nonlinear interaction curve is utilised for the combined loading resistance of class 1 and 2 cross-sections,

$$M_{Ed} \leq M_{N,Rd} = M_{pl,Rd} (1 - n^{1.7}) \quad (5.3)$$

where M_{Ed} is the design ultimate bending moment determined from Equation 5.2, $M_{N,Rd}$ is the design plastic moment resistance reduced due to the applied axial force N_{Ed} , $M_{pl,Rd}$ is the plastic moment capacity (the product of the plastic section modulus W_{pl} and $\sigma_{0.2}$) and $n = N_{Ed}/N_{pl,Rd}$, with N_{Ed} being the design ultimate axial load and $N_{pl,Rd}$ being the design plastic resistance of the gross cross-section (the product of the gross cross-sectional area A and the 0.2% proof stress $\sigma_{0.2}$). For class 3 and 4 cross-sections the design expression is a linear summation of the separate axial and bending utilisation ratios with a limit of unity,

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{Ed}}{M_{Rd}} \leq 1 \quad (5.4)$$

where N_{Rd} is the pure compression design resistance and M_{Rd} is the pure bending design resistance. For class 3 cross-sections, N_{Rd} is the product of the gross cross-sectional area A and the 0.2% proof stress $\sigma_{0.2}$, while for class 4 cross-sections, the effective cross-sectional area A_{eff} is utilised. For class 3 cross-sections M_{Rd} is the elastic moment capacity M_{el} (the product of the elastic section modulus W_{el} and $\sigma_{0.2}$) while for slender class 4 cross-sections the design capacity is the product of the effective section modulus W_{eff}

and $\sigma_{0.2}$. The effective area A_{eff} and effective section modulus W_{eff} can be determined using the slender cross-section design formula from BS 5950-1 (2000) with updated EN 1993-1-4 (2015) class 3 slenderness limits (Chan and Gardner, 2008b; Buchanan et al., 2016a), as reproduced previously as Equation 3.8 in Section 3.4.6 for A_{eff} . The experimental results normalised by their full cross-sectional capacities are shown in Figures 5.12 and 5.13 along with the EN 1993-1-4 (2015) linear interaction curve.

EN 1993-1-4 (2015) does not currently have guidance for cross-section classification of CHS under combined loading. Linear interpolation between the compressive and bending class 3 slenderness limits was therefore used,

$$\frac{D}{t\epsilon^2} = 185 - 95\psi \quad (5.5)$$

with ψ being the ratio of the difference and sum of the compressive σ_c and bending σ_b stresses at the ultimate load,

$$\psi = \frac{\sigma_c - \sigma_b}{\sigma_c + \sigma_b}, \quad (5.6)$$

which is similar to the approach used for EHS by Gardner et al. (2011a). The 80×1.5 and 101.6×1.5 CHS cross-sections are both class 3 under combined loading for the ratios of axial load to bending moment employed in the eccentrically loaded tests.

Figures 5.12 and 5.13 show that the current Eurocode 3 design guidance offers safe side predictions for class 3 ferritic stainless steel CHS under combined axial loading and bending moment, although it is rather conservative, indicating that additional structural efficiency may be sought.

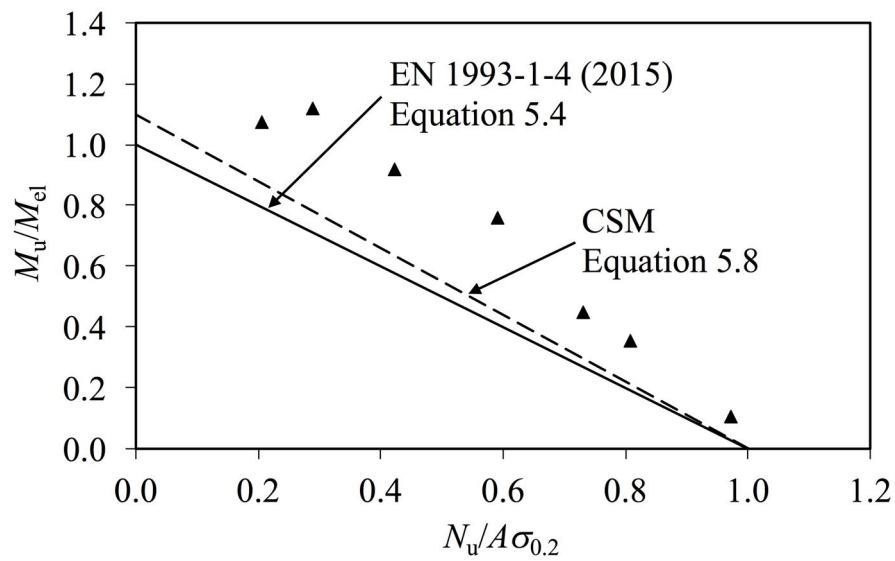


Figure 5.12: Comparison between the 80×1.5 CHS cross-section test results under combined loading with the interaction curves.

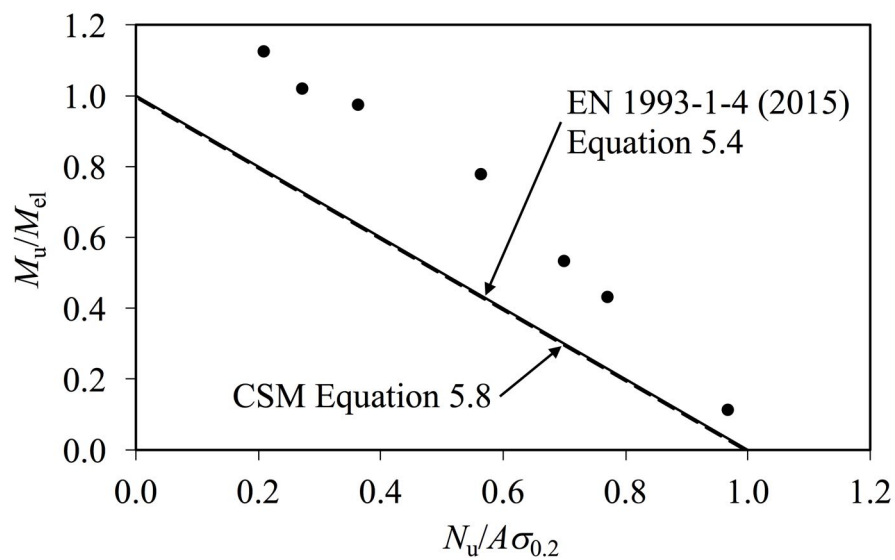


Figure 5.13: Comparison between the 101.6×1.5 CHS cross-section test results under combined loading with the interaction curves.

5.4.2 The continuous strength method design provisions

The continuous strength method (CSM) has been developed to capture the observed characteristics of metallic structural elements better. Cross-section classification is replaced with a continuous relationship between cross-section slenderness and deformation capacity, called the base curve, reflecting the continuous nature of cross-section capacity varying with local slenderness. A strain hardening material model is also adopted, representing the observed behaviour in material tests, with an increase in strength above the yield strength with increasing plastic deformation. The development of this design method was discussed in Section 2.3.2. The CSM was extended to cover non-slender and slender structural carbon steel, stainless steel and aluminium CHS in pure compression and bending in Chapter 3 and for non-slender stainless steel CHS under combined axial loading and bending moment by Zhao et al. (2015b).

The local slenderness of the CHS cross-section is determined from Equation 3.1 in Chapter 3, subsequent equations are also taken from this chapter. The allowable deformation capacity $\varepsilon_{\text{csm}}/\varepsilon_y$, also referred to as the strain ratio, can be calculated from the base curves for non-slender and slender cross-sections with Equations 3.23 and 3.24 respectively, where $C_1 = 0.40$, depending upon whether the local slenderness $\bar{\lambda}_c$ of the cross-section is below or above the yield slenderness limit of $\bar{\lambda}_c = 0.3$. The CSM limiting material stress f_{csm} can be calculated from Equations 3.25 and 3.26, depending upon whether the strain ratio is less than or greater than unity. The ferritic stainless steel strain hardening material model was developed by Bock et al. (2015b) where the material strain ε_u corresponding to the ultimate tensile stress f_u can be calculated from Equation 3.38 and the strain hardening modulus E_{sh} predicted using Equation 3.37, with $C_2 = 0.45$. The CSM axial resistance N_{csm} can then be determined using Equations 3.39 and 3.40 for non-slender and slender cross-sections respectively. The CSM bending resistance M_{csm} can be calculated for non-slender sections from the simplified design expression provided as Equation 3.57 and for slender sections using Equation 3.58.

In the extension of the CSM to cover non-slender stainless steel CHS, Zhao et al. (2015b) adopted the CSM cross-sectional compression and bending resistances as the end points of the linear interaction curve. A nonlinear interaction curve based on the theoretical fully plastic interaction relationship for CHS under combined axial compression and bending moment (adopted by EN 1993-1-4 (2015) and reproduced here as Equation 5.3) from Rondal et al. (1992) was also proposed below a local slenderness transition point of $\bar{\lambda}_c = 0.27$, reverting to the linear interaction curve for compatibility with the elastic end points as the slenderness approaches the yield slenderness limit of $\bar{\lambda}_c = 0.3$. The nonlinear and linear interaction curves are reproduced as

$$M_{Ed} \leq M_{R,csm} = 1.04M_{csm} (1 - n_{csm}^{1.7}) \leq M_{csm} \text{ for } \bar{\lambda}_c \leq 0.27 \quad (5.7)$$

and

$$\frac{N_{Ed}}{N_{csm}} + \frac{M_{Ed}}{M_{csm}} \leq 1 \text{ for } \bar{\lambda}_c > 0.27 \quad (5.8)$$

respectively, where $n_{csm} = N_{Ed}/N_{csm}$.

The 80×1.5 CHS cross-section has a local slenderness of $\bar{\lambda}_c = 0.29$ and thus Equations 3.39 and 3.57 are employed to determine the CSM axial and bending resistances, respectively, while Equation 5.8 is used to define the interaction. The 101.6×1.5 CHS cross-section has a local slenderness of $\bar{\lambda}_c = 0.31$ and the slender CSM axial and bending resistances, Equations 3.40 and 3.58 respectively, have been applied as the interaction curve end points in Equation 5.8. The CSM interaction curves are plotted in Figures 5.12 and 5.13 along with the experimental data and can be seen to offer improved predictions of the cross-sectional capacities under combined loading for the 80×1.5 CHS cross-section. The application of the CSM compressive and bending end points to the 101.6×1.5 CHS cross-section yields very similar results to EN 1993-1-4 (2015), since the cross-section is only marginally beyond the yield slenderness limit.

5.4.3 EN 1993-1-4 (2015) design provisions for the member behaviour

The current EN 1993-1-4 (2015) design approach for stainless steel beam-columns utilises the same form as the interaction formulae provided for structural carbon steel in EN 1993-1-1 (2014), although the interaction factors have been modified to account for the influence of the nonlinear material response on member instability. The beam-column combined loading design expression is reproduced as follows,

$$\frac{N_{Ed}}{N_{b,Rd}} + k \left(\frac{M_{Ed} + N_{Ed}e_N}{\beta_w W_{pl}\sigma_{0.2}} \right) \leq 1 \quad (5.9)$$

where N_{Ed} is the design ultimate axial load, M_{Ed} is the design bending moment (being N_{Ed} multiplied by $(e_0 + \omega_0)$, which is different to that used at the cross-section level (Equation 5.2), with the contribution from the mid-height lateral deflection at the ultimate load ω_u accounted for in the interaction factor k), $N_{b,Rd}$ is the flexural buckling design resistance, e_N is the shift in the neutral axis (for local buckling of slender cross-sections) when the cross-section is subjected to uniform compression (which is taken as zero for doubly symmetric cross-sections), and β_w is a factor which modifies the expression for the various cross-section classes — for class 1 and 2 cross-sections $\beta_w = 1.0$, for class 3 cross-sections $\beta_w = W_{el}/W_{pl}$ and for class 4 cross-sections $\beta_w = W_{eff}/W_{pl}$. The EN 1993-1-4 (2015) interaction factor k can be calculated using

$$1.2 \leq k = 1 + 2 (\bar{\lambda} - 0.5) \frac{N_{Ed}}{N_{b,Rd}} \leq 1.2 + 2 \frac{N_{Ed}}{N_{b,Rd}}. \quad (5.10)$$

The experimental results normalised by their cross-sectional capacities are shown in Figures 5.14 and 5.15 along with the EN 1993-1-4 (2015) member level interaction curves, with the point at which the interaction curves meet the horizontal axis corresponding to the buckling reduction factor χ . Classification under combined loading was again undertaken using Equations 5.5 and 5.6, and both the 80×1.5 and 101.6×1.5 CHS cross-sections were found to be class 3 for the ratios of axial load to bending moment employed in the eccentrically loaded tests; the concentrically loaded 101.6×1.5 -1600-P

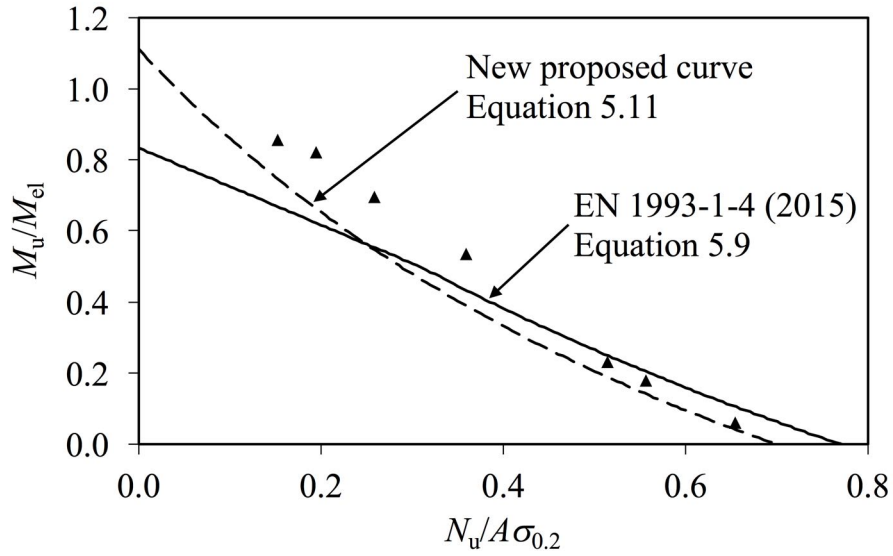


Figure 5.14: Comparison of the 80×1.5 CHS beam-column member test results with the interaction curves.

CHS specimen was classified as class 4 and therefore the effective cross-sectional area A_{eff} and section modulus W_{eff} were used. Figures 5.14 and 5.15 show that the current guidance is conservative towards the bending end point, which mirrors the findings of Zhao et al. (2015b) and is attributed to the neglecting of strain hardening for the bending end point and to conservative interaction factors, although it is unconservative towards the compression end point.

5.4.4 Proposed design provisions for the member behaviour

Improvements to the current EN 1993-1-4 (2015) design approach for stainless steel beam-columns can be focussed on two key areas: the interaction factor and its two end points. The current pure compression end point is the flexural buckling resistance $N_{b,Rd}$, which was previously observed in Chapter 4 to provide unconservative predictions for certain global slenderness values. The pure bending end point is the bending resistance of the cross-section $\beta_w W_{pl} \sigma_{0.2}$, which was seen in Chapter 3 to provide overly conservative capacity predictions for stocky cross-sections. The proposals from these chapters can be used to provide more suitable end points; the new flexural buckling curve can be used to

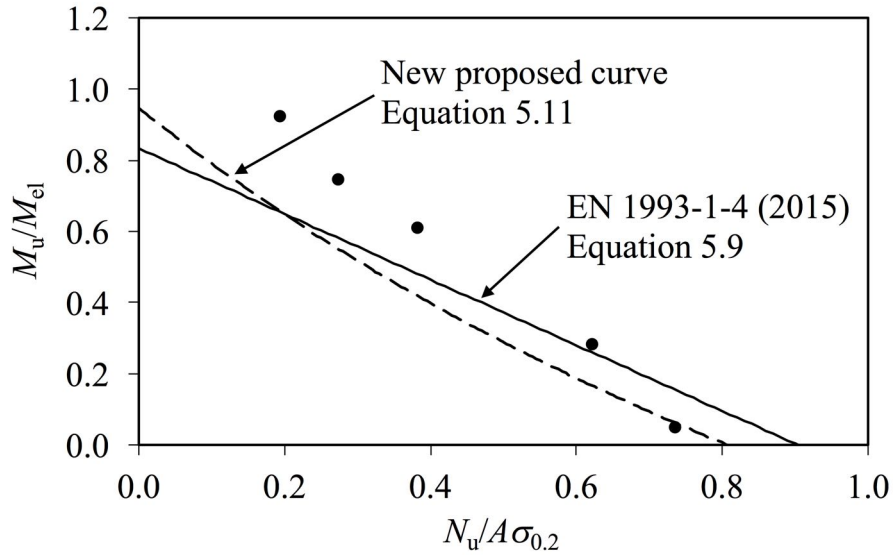


Figure 5.15: Comparison of the 101.6×1.5 CHS beam-column member test results with the interaction curves.

predict a more appropriate flexural buckling resistance $N_{b,Rd}$, while the CSM bending resistance M_{csm} allows for greater bending capacities to be attained through the base curve and material strain hardening. An advantage of improved end points is that the interaction factor does not need to compensate for their inaccuracies and can therefore be more efficient. The new proposed beam-column design expression is as follows,

$$\frac{N_{Ed}}{N_{b,Rd}} + k_{csm} \frac{M_{Ed}}{M_{csm,Rd}} \leq 1 \quad (5.11)$$

where k_{csm} is the new interaction factor, which is determined using the same procedure as Greiner and Lindner (2006), Boissonnade et al. (2006) and Zhao et al. (2016a). Individual k_{csm} factors are calculated for the experimental and numerical dataset using

$$k_{csm} = \left(1 - \frac{N_{Ed}}{N_{b,Rd}}\right) \frac{M_{csm,Rd}}{M_{Ed}}, \quad (5.12)$$

which is a rearrangement of Equation 5.11, with simplified formulae then fitted to the data for a specific axial compressive load level $n = N_{Ed}/N_{b,Rd}$.

Data for seven compressive load levels with $n = 0.2$ to $n = 0.8$, in steps of 0.1, were isolated from the full dataset and the k_{csm} interaction factor values plotted with varying

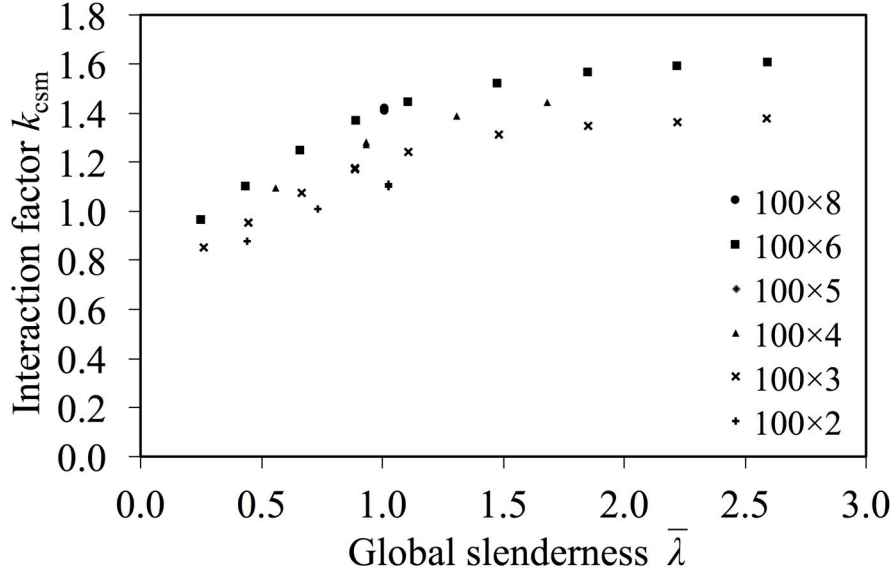


Figure 5.16: k_{csm} factors determined from the numerical dataset for the austenitic specimens with a load level of $n = 0.2$.

global slenderness $\bar{\lambda}$, as shown in Figure 5.16 for the austenitic dataset with $n = 0.2$. The simplified design expression for k_{csm} is taken as the typical two-stage linear piecewise form (Greiner and Lindner, 2006; Boissonnade et al., 2006; Zhao et al., 2016a),

$$k_{\text{csm}} = 1 + D_1 (\bar{\lambda} - D_2) n \leq 1 + D_1 (D_3 - D_2) n \quad (5.13)$$

where D_1 and D_2 are coefficients defining the linear relationship between the k_{csm} interaction factor and global slenderness $\bar{\lambda}$ in the $\bar{\lambda} \leq D_3$ low slenderness range, beyond which the k_{csm} interaction factor is a constant value $(1 + D_1 (D_3 - D_2) n)$ with increasing global slenderness. The D_1 and D_2 coefficients for the discrete compressive load levels were calculated from a regression fit between the upper bound of the assembled dataset and Equation 5.13 for $0.2 \leq \bar{\lambda} \leq 1.2$; the final D_1 and D_2 coefficients are averaged values for all of the compressive load levels and are listed in Table 5.18 for the three types of stainless steel used in construction. The D_3 coefficients are determined from fitting Equation 5.13 to the upper bound of the dataset for $n \leq 0.4$, and are also provided in Table 5.18.

The calculated and proposed k_{csm} interaction factors are plotted for the austenitic dataset

Table 5.18: Proposed stainless steel CHS interaction curve coefficients.

Type	D_1	D_2	D_3
Austenitic	2.5	0.30	1.3
Duplex	2.0	0.38	1.3
Ferritic	1.9	0.35	1.3

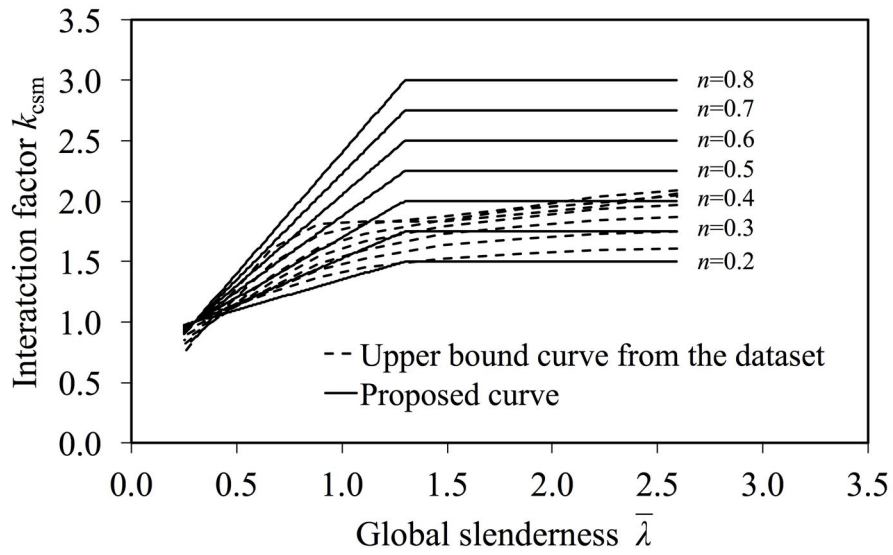


Figure 5.17: Proposed and calculated k_{csm} interaction factors for the austenitic beam-column specimens.

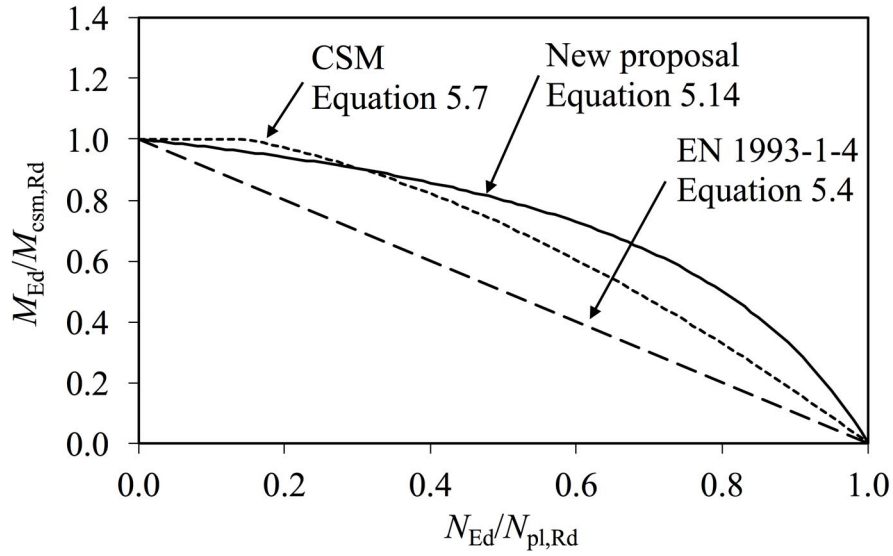


Figure 5.18: Comparison of the interaction curves at the cross-section level.

in Figure 5.17, with a dashed line passing through the upper bound of the calculated k_{csm} values from the austenitic dataset for the various axial load levels n . It is clear from Figure 5.17 that there is a large difference between the proposed k_{csm} interaction factor and the calculated value for the assembled dataset for high axial compressive load levels n and high $\bar{\lambda}$ global slenderness values; for members with high slenderness and high axial load level, the structural response is controlled by column buckling, rather than the bending resistance, and therefore this disparity is acceptable (Greiner and Lindner, 2006; Greiner and Kettler, 2008; Zhao et al., 2016a), and the resulting errors are small.

When the global slenderness tends to zero the new proposed beam-column expression becomes

$$\frac{N_{\text{Ed}}}{N_{\text{pl,Rd}}} + \left(1 - 0.75 \frac{N_{\text{Ed}}}{N_{\text{pl,Rd}}}\right) \frac{M_{\text{Ed}}}{M_{\text{csm,Rd}}} \leq 1 \quad (5.14)$$

for the austenitic dataset, where $N_{\text{pl,Rd}}$ is the yield load. This expression is similar to the CSM interaction curve presented previously as Equation 5.7, as shown in Figure 5.18 for a cross-section on the class 3 limit.

The new proposed interaction factor k_{csm} and end points are plotted in Figures 5.14

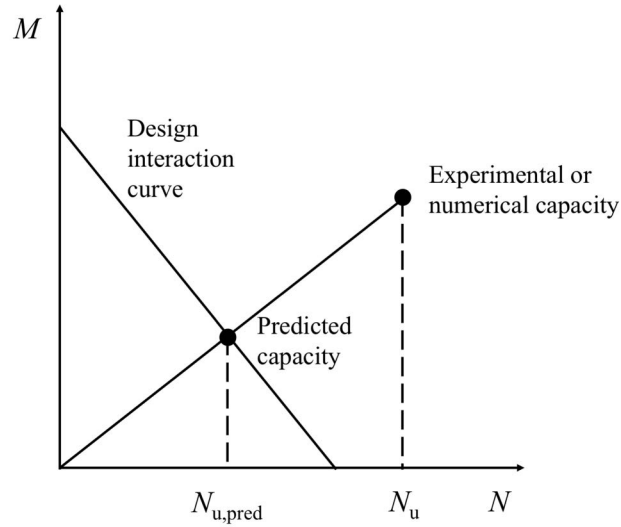


Figure 5.19: Definition of $N_{u,pred}$ on the moment-axial interaction curve.

and 5.15. The benefit from utilising the increased CSM bending resistance M_{csm} and the more appropriate pure compression end point is apparent; the experimental results are now closer to and typically above the proposed interaction curve, particularly towards the pure compression end.

5.4.5 Discussion of existing and new design provisions

The existing and proposed design approaches for stainless steel CHS under combined loading at the local and global level can be appraised using the assembled experimental and numerical dataset. The predicted capacity from the various design methods can be calculated assuming proportional loading, as defined in Figure 5.19.

At the cross-section level, the average ultimate compressive capacity normalised by the Eurocode and CSM design capacity, $N_u/N_{u,EC3}$ and $N_u/N_{u,csm}$ respectively, is provided in Table 5.19. The 80×1.5 -350-P-100E CHS result is omitted from the results in Table 5.19, due to the end plates yielding in bending. The EN 1993-1-4 (2015) and CSM interaction

Table 5.19: Comparison of the ultimate and predicted combined axial load and bending strength.

Cross-section	Mean		COV	
	$N_u/N_{u,EC3}$	$N_u/N_{u,CSM}$	$N_u/N_{u,EC3}$	$N_u/N_{u,CSM}$
80×1.5	1.25	1.19	0.10	0.07
101.6×1.5	1.26	1.27	0.08	0.08
All test specimens	1.26	1.23	0.09	0.08

curves are plotted with the experimental data in Figures 5.12 and 5.13. The current Eurocode 3 design guidance offers rather conservative predictions of the combined axial loading and bending moment capacity, while the CSM offers on average more accurate and less scattered resistance predictions, reflected by lower mean and coefficient of variation (COV) values. The 101.6×1.5 CHS cross-section resistance is slightly more accurately predicted by the existing EN 1993-1-4 (2015) approach due to the cross-section being classified as class 3 under combined loading, whereas for the CSM it is classified as a slender cross-section and is therefore predicted to attain a slightly lower resistance.

The average ultimate compressive capacity normalised by the Eurocode and new proposed member level resistance predictions, $N_u/N_{u,EC3}$ and $N_u/N_{u,proposed}$, are provided in Table 5.20 for the new and existing CHS experimental specimens, including the austenitic beam-column results from Talja (1997) and Zhao et al. (2016c), and in Table 5.21 for the full experimental and numerical dataset. The EN 1993-1-4 (2015) and new proposed interaction curves are plotted with the ferritic CHS experimental beam-column data in Figures 5.14 and 5.15. The experimental only dataset from Table 5.20 shows that the new proposed end points and interaction factor result in less scattered, but also slightly less accurate predictions, than the existing EN 1993-1-4 (2015) approach, although the average $N_u/N_{u,proposed}$ values for all of the individual cross-sections are greater than unity and therefore safe with the new proposals. The apparent reduced accuracy is attributed to the more conservative compressive end point, though it can be seen from Figures 5.14 and 5.15 that towards the pure compression end, the existing interaction curve results in a number of unconservative predictions, which reduces the average normalised load.

Table 5.20: Comparison of the ultimate and predicted beam-column strengths for the new and existing test specimens.

Cross-section	Source	Type	Mean		COV	
			$N_u/N_{u,EC3}$	$N_u/N_{u,proposed}$	$N_u/N_{u,EC3}$	$N_u/N_{u,proposed}$
140×2	Talja (1997)	A	1.14	1.27	0.03	0.01
140×4	Talja (1997)	A	1.12	1.24	0.07	0.03
60.5×2.8	Zhao et al. (2016c)	A	1.00	1.07	0.02	0.02
76.3×3	Zhao et al. (2016c)	A	0.97	1.03	0.04	0.02
80×1.5	This study	F	1.09	1.10	0.12	0.05
101.6×1.5	This study	F	1.13	1.19	0.13	0.07
Average	-	-	1.06	1.13	0.10	0.08

Table 5.21: Comparison of the ultimate and predicted beam-column strengths for the full dataset.

Type	Mean		COV	
	$N_u/N_{u,EC3}$	$N_u/N_{u,proposed}$	$N_u/N_{u,EC3}$	$N_u/N_{u,proposed}$
Austenitic	1.12	1.10	0.10	0.06
Duplex	1.18	1.12	0.09	0.05
Ferritic	1.17	1.10	0.12	0.05
Average	1.15	1.11	0.11	0.05

The new proposal for stainless steel CHS beam-columns can be seen to be more accurate and consistent (with a substantially lower COV) in its resistance predictions than the current EN 1993-1-4 (2015) approach for the full experimental and numerical dataset in Table 5.21. The reduced COV is particularly beneficial in the reliability analysis in Section 5.5.

A further comparison can be made between the existing EN 1993-1-4 (2015) approach and the new proposal by plotting the ultimate compressive capacity normalised by the predicted value against the angle parameter θ , as shown in Figures 5.21 to 5.26. The angle parameter θ is defined as

$$\theta = \tan^{-1} \left(\frac{N_{Ed}/N_R}{M_{Ed}/M_R} \right), \quad (5.15)$$

where N_R and M_R are the predicted pure compression and bending member strengths, with $\theta = 0^\circ$ corresponding to pure bending and $\theta = 90^\circ$ being pure compression, as shown in Figure 5.20. It is again apparent that the new proposal provides more efficient

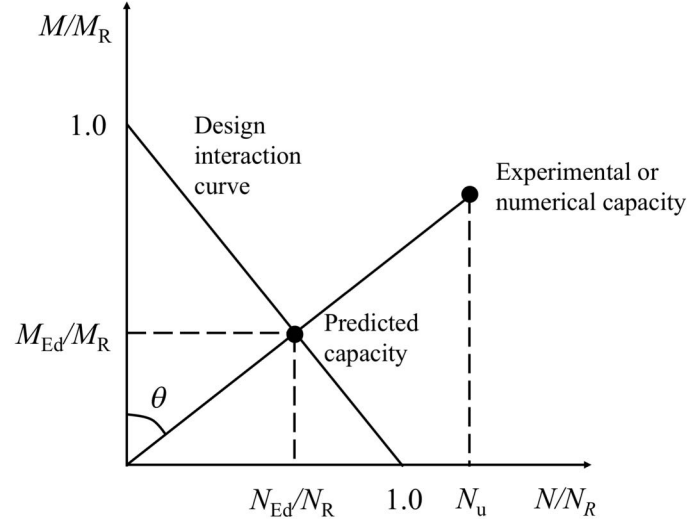


Figure 5.20: Definition of θ on the moment-axial interaction curve.

predictions of the compressive resistance than the current EN 1993-1-4 (2015) method.

5.5 Reliability analyses

Reliability analyses to determine the required partial factors were previously introduced and calculated in Section 4.6.

The new experimental and numerical dataset produced in this study has been used to reassess the γ_{M1} partial factor with the current EN 1993-1-4 (2015) CHS beam-column design approach, as summarised in Table 5.22, where n is the size of the dataset, the over-strength factor is the mean yield strength normalised by the nominal yield strength, V_{fy} is the coefficient of variation of the yield strength, V_{geometry} is the coefficient of variation of the geometry of the cross-section, $k_{d,n}$ is the fractile factor and is related to the size of the dataset, b is average ratio of the experimental to model resistance using a least squares fit and V_{δ} is the coefficient of variation of the tests relative to the resistance model. As in Chapter 4 the over-strength factors, V_{fy} and V_{geometry} used in the statistical analysis were

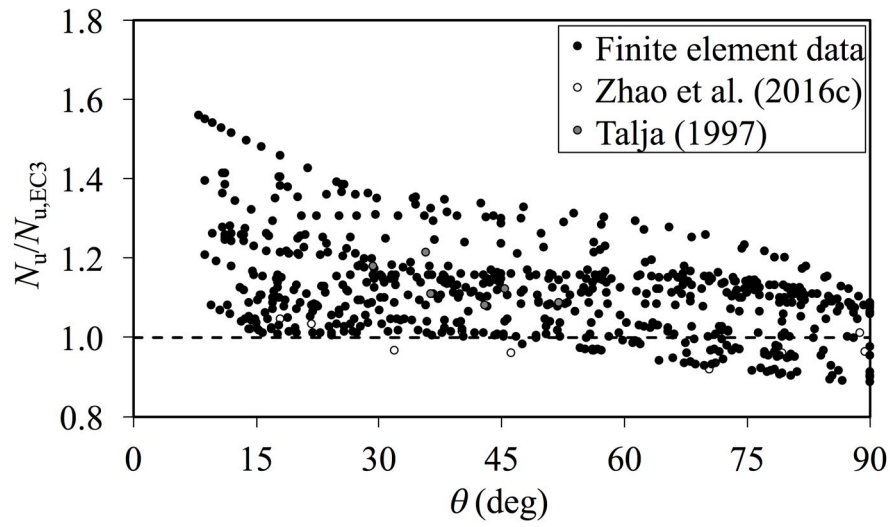


Figure 5.21: Normalised resistance predictions for the austenitic stainless steel CHS beam-column dataset with the EN 1993-1-4 (2015) approach.

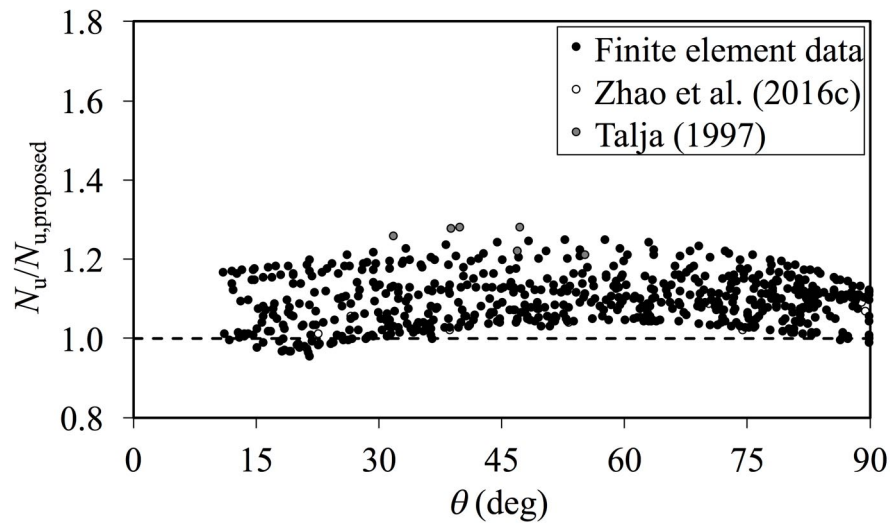


Figure 5.22: Normalised resistance predictions for the austenitic stainless steel CHS beam-column dataset with the new proposed approach.

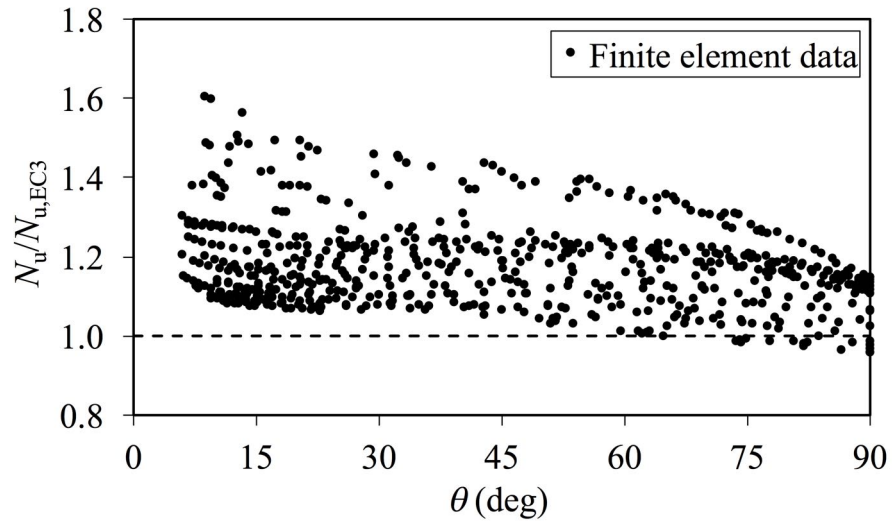


Figure 5.23: Normalised resistance predictions for the duplex stainless steel CHS beam-column dataset with the EN 1993-1-4 (2015) approach.

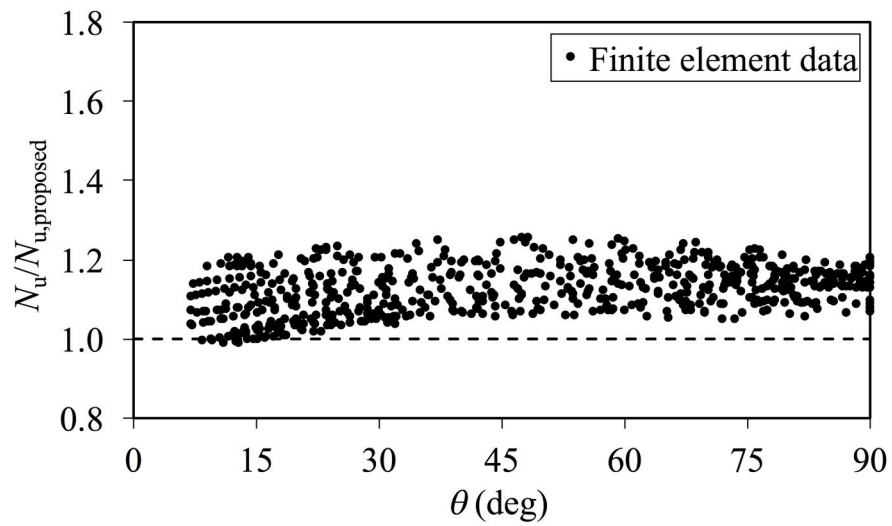


Figure 5.24: Normalised resistance predictions for the duplex stainless steel CHS beam-column dataset with the new proposed approach.

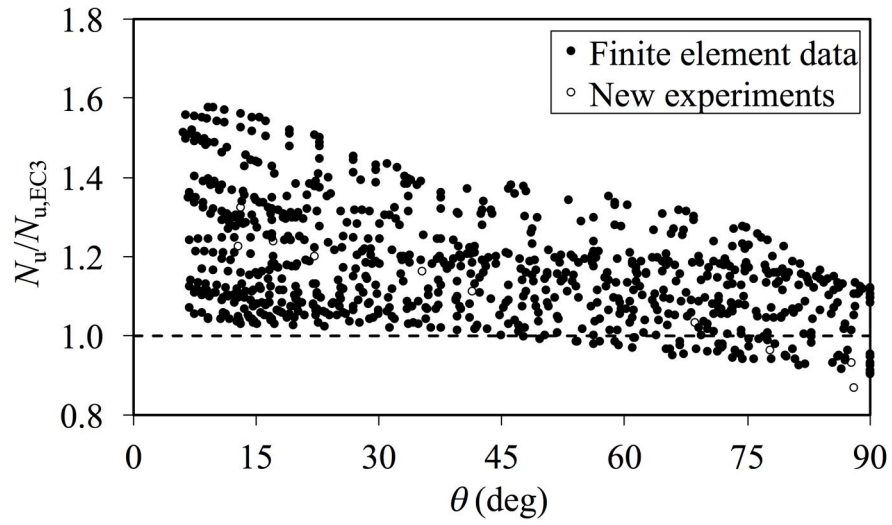


Figure 5.25: Normalised resistance predictions for the ferritic stainless steel CHS beam-column dataset with the EN 1993-1-4 (2015) approach.

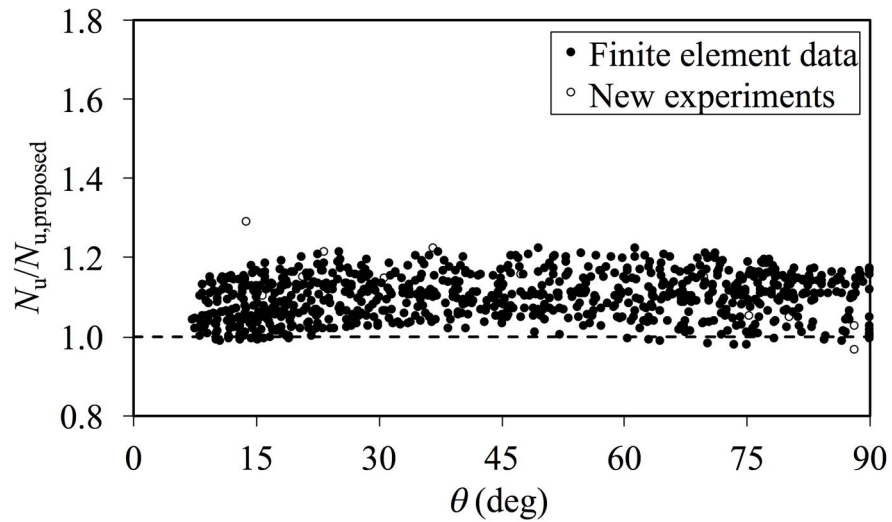


Figure 5.26: Normalised resistance predictions for the ferritic stainless steel CHS beam-column dataset with the new proposed approach.

taken from Afshan et al. (2015). The required γ_{M1} partial factor for the new proposed end points and interaction curve are provided in Table 5.23. The new proposal is more consistent across all three types of stainless steel with lower V_δ coefficients than the EN 1993-1-4 (2015) approach. Overall it can be seen that for both the existing and proposed design approaches the required level of reliability is achieved with safety factors lower than the required value of $\gamma_{M1} = 1.1$.

5.6 Conclusions

A comprehensive experimental programme focusing upon ferritic stainless steel circular hollow section members under combined axial loading and bending moment has been undertaken. The investigation consisted of material property tests, stub column tests, cross-section level combined axial loading and bending moment tests and member level beam-column tests on ferritic stainless steel CHS. The experimental data were complemented by a pool of finite element results generated through a parallel numerical study. The ultimate loads were then compared against the current European design guidance and it was observed that the current design approach is conservative at the cross-section level and for members with a high ratio of bending moment to axial load, while it is slightly unconservative for members with a low ratio of bending moment to axial load. A more efficient deformation based design approach, the continuous strength method (CSM), was generally found to offer improvements over the current European guidance at the cross-section level. A new interaction curve is proposed at the member level with a more efficient bending resistance end point and a more appropriate compressive capacity end point, based on previous work in Chapters 3 and 4, and is observed to offer improved stainless steel CHS beam-column resistances. Statistical analyses were undertaken for both the current EN 1993-1-4 (2015) and new proposed design approaches, and both were found to satisfy the requirements for the incumbent partial safety factor for γ_{M1} of 1.1 .

Table 5.22: Summary of the statistical analysis results for the EN 1993-1-4 (2015) stainless steel CHS beam-column design approach.

Type	Dataset	n	Over-strength	V_{f_y}	V_{geometry}	$k_{d,n}$	b	V_{δ}	γ_{M1}
A	Tests	18	1.3	0.060	0.05	3.09	1.057	0.007	0.93
A	Tests and FE	612	1.3	0.060	0.05	3.09	1.017	0.055	1.02
D	Tests	-	-	-	-	-	-	-	-
D	Tests and FE	621	1.1	0.030	0.05	3.09	1.071	0.049	1.08
F	Tests	12	1.2	0.045	0.05	3.09	0.997	0.009	1.03
F	Tests and FE	813	1.2	0.045	0.05	3.09	1.041	0.073	1.09

Table 5.23: Summary of the statistical analysis results for the new proposed stainless steel CHS beam-column design approach.

Type	Dataset	n	Over-strength	V_{f_y}	V_{geometry}	$k_{d,n}$	b	V_{δ}	γ_{M1}
A	Tests	18	1.3	0.060	0.05	3.09	1.156	0.008	0.85
A	Tests and FE	612	1.3	0.060	0.05	3.09	1.047	0.031	0.95
D	Tests	-	-	-	-	-	-	-	-
D	Tests and FE	621	1.1	0.030	0.05	3.09	1.097	0.030	1.01
F	Tests	12	1.2	0.045	0.05	3.09	1.089	0.005	0.94
F	Tests and FE	813	1.2	0.045	0.05	3.09	1.052	0.032	1.00

Chapter 6

Additive manufacturing

6.1 Introduction

Metallic additive manufacturing is becoming more commonplace in high added value industries, such as the aerospace, automotive, defence and medical fields, due to its numerous advantages over traditional manufacturing techniques. Additive manufacturing allows for more geometric freedom, the ability to produce small production runs at low cost and with reduced material waste, which is particularly important when the feedstock, such as titanium or stainless steel powder, is expensive. These advantages are also true for the construction industry, although the raw material costs tend to be lower due to the use of cheaper materials, and consequently additive manufacturing is only starting to be considered as a viable manufacturing technique in construction. As discussed in the literature review in Section 2.5.3, to date the majority of additive manufacturing research has focussed on the cementitious methods and plastic construction, with limited focus on metallic structures, from a construction viewpoint. There has been significant effort from a material science standpoint, as reviewed previously in Section 2.5.2, with extensive work being carried out on the metallic additive manufacturing production processes and basic material properties, but there has been very limited research to date into potential structural engineering applications. The aim of this chapter is to investigate further the directionality of powder bed fusion stainless steels in both tension

and compression and to undertake cross-sectional tests to provide initial experimental data to appraise the applicability of existing design methods to sections produced through this novel manufacturing route. The work in this chapter has been presented previously by Buchanan et al. (2016b) and Buchanan et al. (2017).

6.2 Experimental investigation

The experimental investigation consisted of tensile and compressive material coupon tests and compressive tests on square hollow section (SHS) stub columns. Two different stainless steel grades were studied — a precipitation hardening martensitic grade PH1 (also known as 15-5 PH, EN 1.4540 and X4CrNiCuNb164), commonly used for aerospace components and parts for corrosive high pressure environments (Khalid Rafi et al., 2013; EOS, 2015), and an austenitic grade 316L (also referred to as EN 1.4404 and X2CrNiMo17-12-2), which is widely used for aerospace, automotive, chemical, construction, consumer and nuclear applications (EOS, 2014). Tensile material properties in a variety of building orientations were determined for both stainless steel grades, with compressive material properties measured for the austenitic 316L grade only. The cross-sections were built vertically, with their longitudinal axis perpendicular to the building layers. The cross-sectional behaviour was examined for the 316L material as this is a common grade of stainless steel used in structural engineering and is included in the European stainless steel design standard EN 1993-1-4 (2015). The chemical composition of the metallic powders from the manufacturer's datasheet (EOS, 2015, 2014) are provided in Table 6.1, with Nb being Niobium and the remaining element symbols defined in Section 2.2.2. The tensile and compressive coupons were produced at Lappeenranta University of Technology (LUT) and the Fraunhofer Institute for Laser Technology (ILT) and the stub columns were manufactured by EOS Finland, while the experiments were undertaken at Imperial College London (ICL). The manufacturing method, including processing parameters, and the experiments are described in this

section.

6.2.1 Powder bed fusion manufacturing

The coupons and stub columns were built using powder bed fusion (PBF), with equipment and powder from EOS who market their manufacturing process as direct metal laser sintering (DMLS). Development of PBF methods started in the late 1980s (Levy et al., 2003) and accelerated during the late 1990s in Germany (Mertens et al., 2014; Vilaro et al., 2011) with the first patent to turn metal powder into metal components filed in 1997 (Yap et al., 2015). The first attempts at stainless steel PBF, which used 316L grade powder, were unsuccessful due to balling effects (Abe et al., 2001); however, by 2010 the process had advanced such that 316L PBF had reached relative densities approaching 100% (Tolosa et al., 2010).

The general laser PBF manufacturing process is outlined in Figure 6.1. First, the part is drawn in CAD software as a three-dimensional solid model; it is then converted into an STL model, which describes the surface geometry using tessellated triangles; and finally it is turned into two-dimensional slices that represent the individual layers to be built. The components tested in this study were initially drawn and converted to an STL model in SolidWorks CAD software (Dassault Systèmes, 2015b) with the two-dimensional slicing undertaken using Netfabb software (Autodesk, 2015b). These layers are then sent to the PBF machine with appropriate processing parameters and the building process can begin. The recoating mechanism spreads powder across the building platform and then a laser beam scans the two-dimensional sliced cross-section (shown in Figure 6.2) selectively melting and fusing together the first layer of powder to the building platform. The building platform then lowers by the layer thickness and the build cycle repeats until the part has been built layer by layer. The unused powder is removed and sieved to be used in the next build job. The build platform, with built parts still attached, can be taken out of the machine (Figures 6.3 and 6.4), the parts heat treated if required and then

Table 6.1: Chemical composition from the manufacturer's datasheet.

Material	Cr (%)	Ni (%)	Cu (%)	Mn (%)	Si (%)	C (%)	Mo (%)	Nb (%)	P (%)	S (%)	N (%)	Fe (%)
PH1	14.0 -15.5	3.5 - 5.5	2.5 - 4.5	≤1.00	≤1.00	≤0.07	≤0.50	0.15 - 0.45	-	-	-	Balance
316L	17.0 -19.0	13.0 -15.0	0.5	2	0.75	0.03	2.25 - 3.00	-	0.025	0.01	0.1	Balance

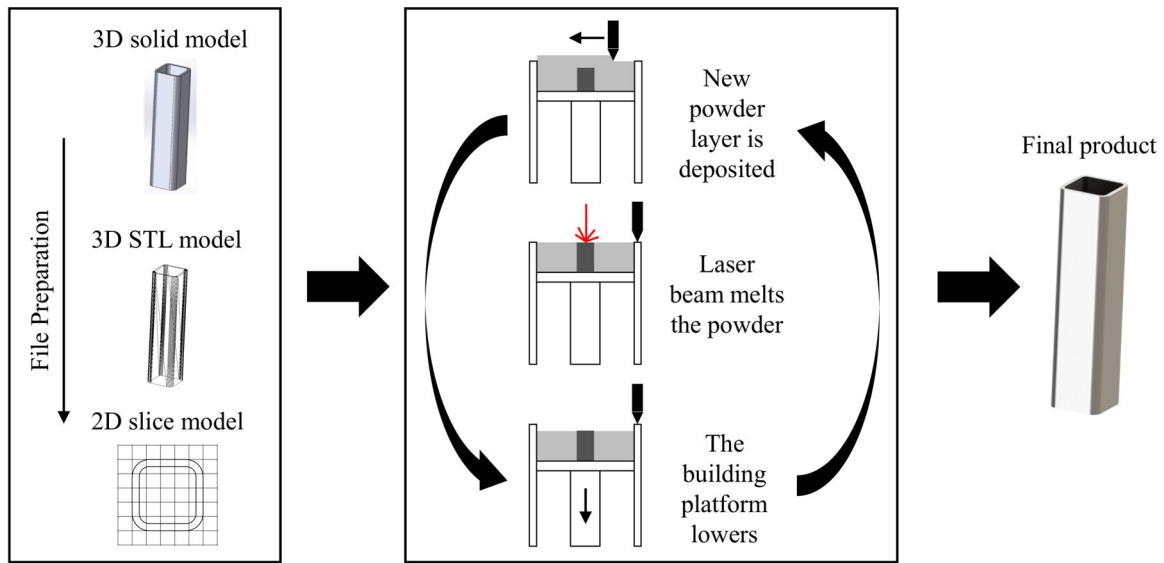


Figure 6.1: Overview of the PBF process from the computer model to the final product.

detached from the building plate.

Support structures, such as those shown in Figure 6.5, are required when building objects with cantilevering elements (both horizontal and below 45°), for mechanical stability (Prashanth et al., 2014) and to dissipate heat into the building platform, due to the unmelted powder's poor heat conductivity, and for slender vertical elements that, if distorted, could be damaged by the recoating mechanism. These structures were automatically produced using Magics software (Materialise, 2015), based upon inputted acceptable cantilevering build angles and the building material.

The PH1 tensile coupons were built from EOS PH1 powder at the LUT laser facilities in Lappeenranta, Finland using a modified research EOS M270 machine. The 316L tensile and compressive coupons were manufactured using EOS 316L powder in an EOS M270 machine at ILT in Aachen, Germany, while the stub columns were built by EOS Finland using an EOS M280 machine. The particle size of both powders was within the 21 to $53\ \mu\text{m}$ range. The processing parameters used were those recommended by the manufacturer, which are optimised for the powder and manufacturing equipment.

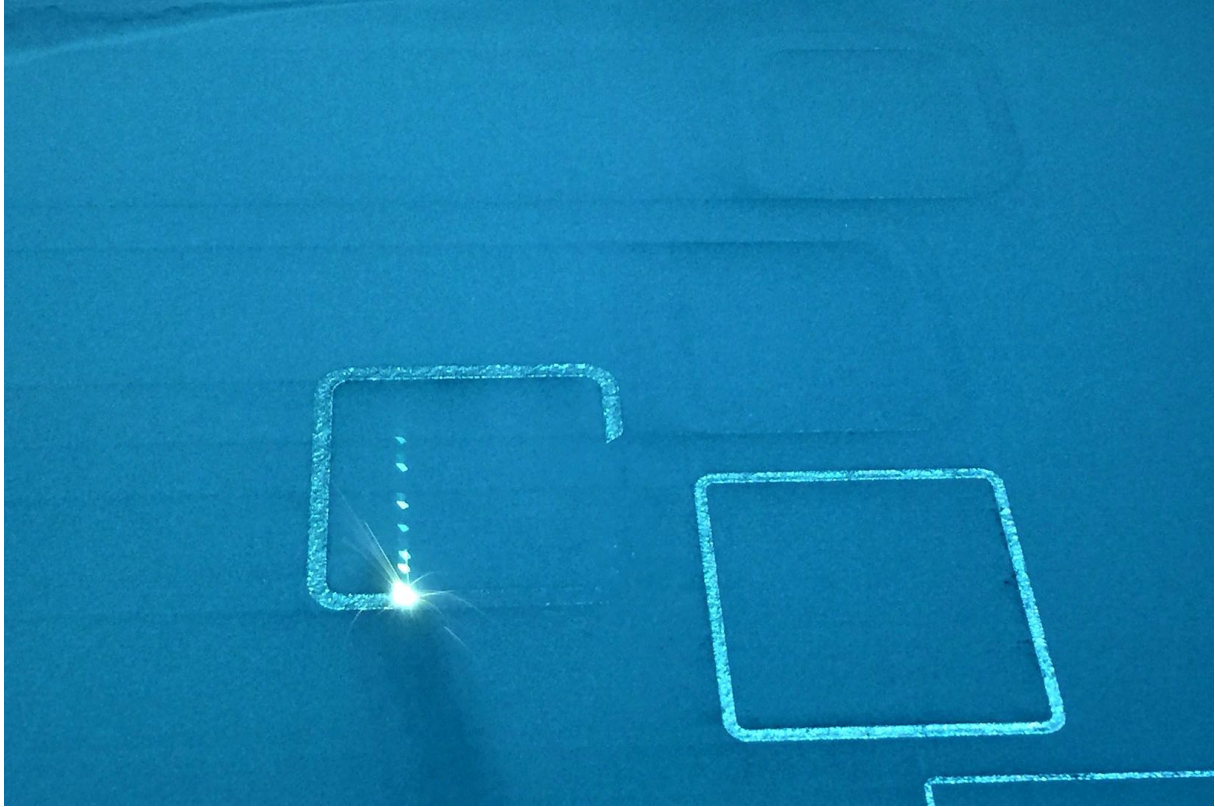


Figure 6.2: The 316L stub columns during building.

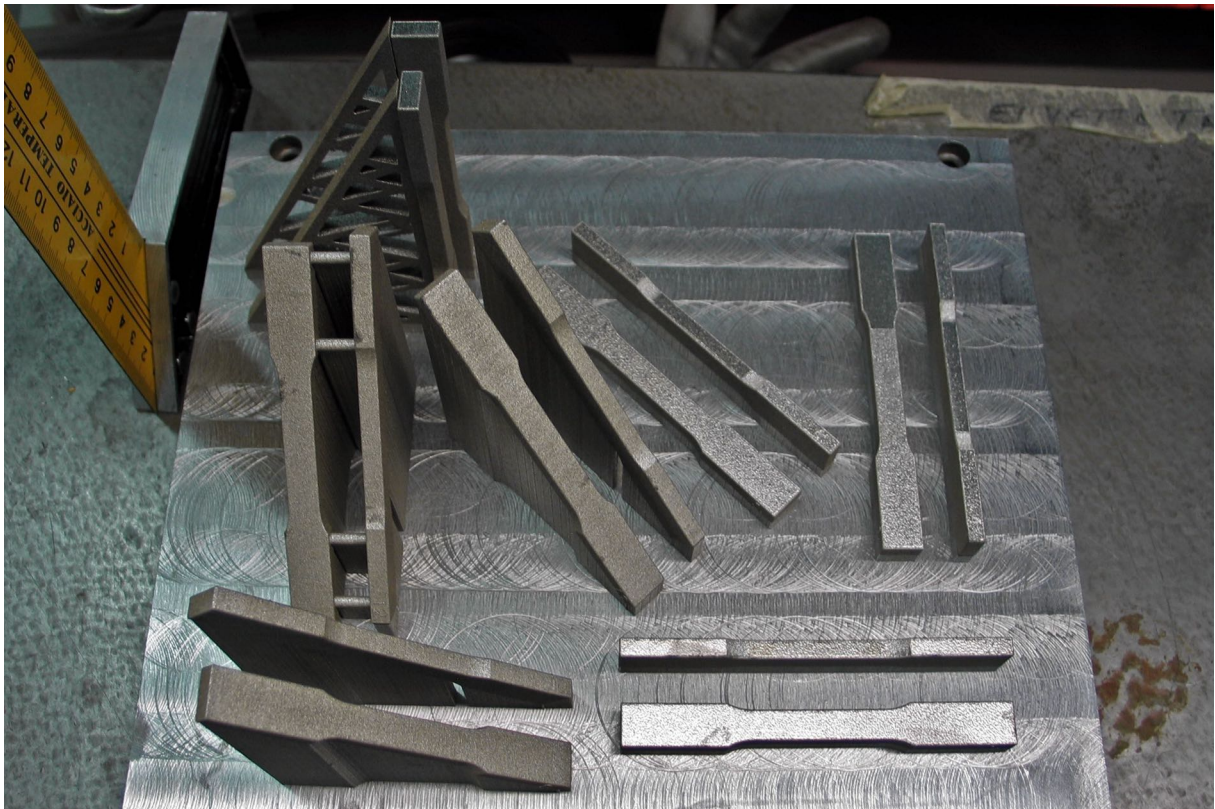


Figure 6.3: The PH1 tensile coupons after building, still attached to the build platform.

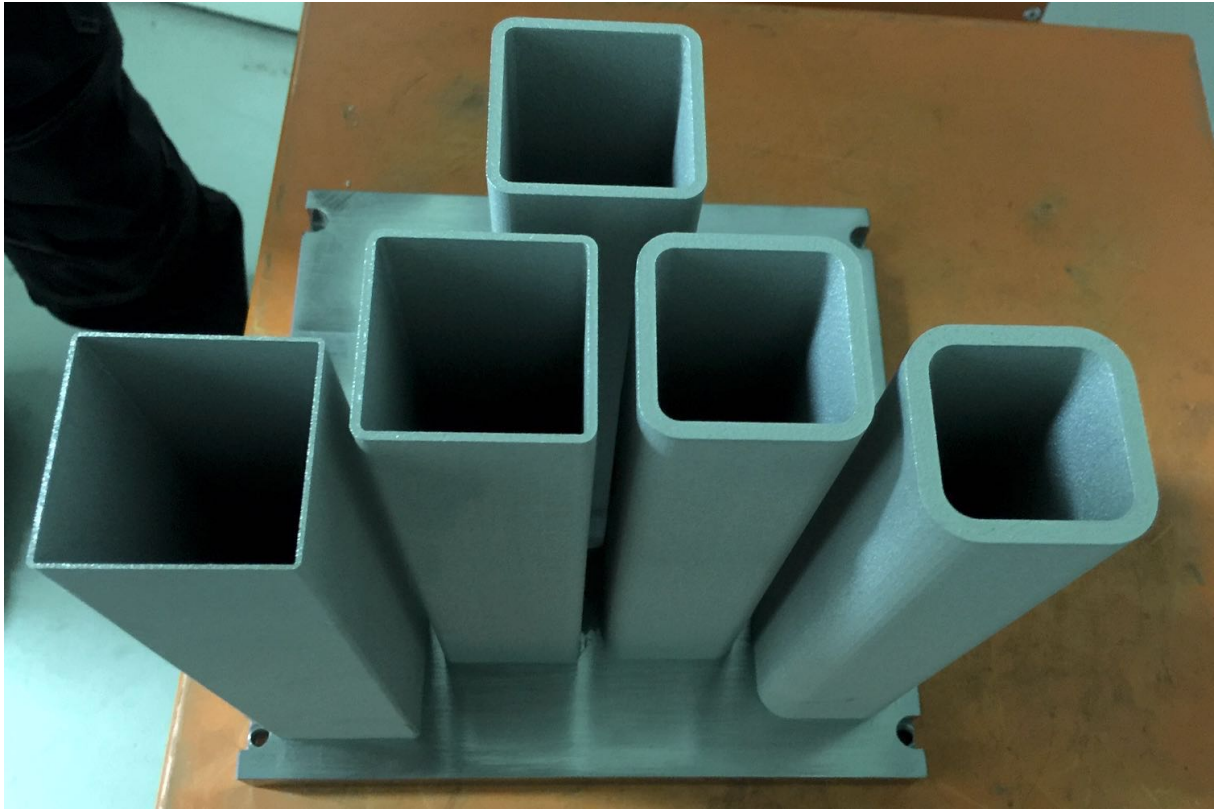


Figure 6.4: The 316L stub columns after building, still attached to the build platform.

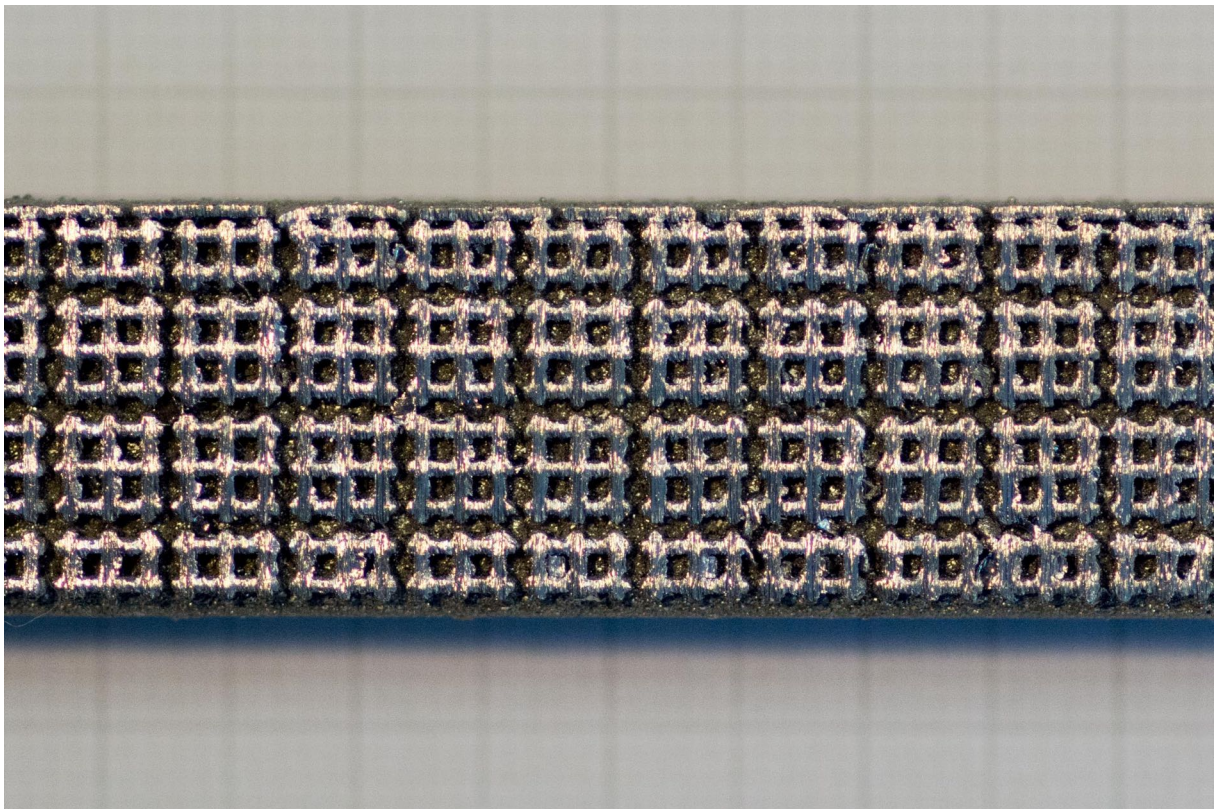


Figure 6.5: A cross-section through a typical support structure for a compression coupon.

The power of the ytterbium fibre laser, laser spot size on the building surface, scanning velocity (laser spot velocity), layer thickness, hatch distance (distance between adjacent scanning tracks), preheated temperature of the build platform to minimise the residual stresses close to the building platform (Vilaro et al., 2011), and the protective gas, to prevent melt pool oxidation and the formation of nitridic phases (Meier and Haberland, 2008; Vilaro et al., 2011), employed are listed in Table 6.2. The scanning strategy (i.e. how the laser spot traverses the build surface) rotated by 45° with each layer to minimise the internal defects (Yadroitsev et al., 2007) and increase the relative density. The coupons and stub columns were all built at the same time within their respective groups, i.e. all of the 316L tensile coupons were built at the same time on the same build platform. The build time was roughly 40 hours for the tensile and compressive coupons and 140 hours for the stub columns with an approximate volumetric build rate of 2 mm³/s. No post building surface treatment was undertaken, apart from removal using a bandsaw from the building platform, grinding off the support structures and machining of the stub column ends.

Prior to testing, the PH1 tensile coupons were X-ray imaged to check for obvious internal imperfections, such as large pores, as shown in Figure 6.6. No obvious imperfections were found and this process was not repeated for the 316L components since the building processes were deemed successful and no defects were expected in the test pieces. The built relative density (the ratio of the attained density to the theoretical achievable density) was not measured, although by following the manufacturer's recommended processing parameters, it is expected to have approached 100%.

6.2.2 Tensile coupon tests

Tensile coupon tests were undertaken to ascertain the tensile engineering stress–strain properties of the material, and to observe any directionality effects arising from the manufacturing method. Fourteen coupons were built in a range of building orientations

Table 6.2: Summary of the adopted processing parameters.

Material	Laser power (W)	Laser spot size (μm)	Scanning velocity (mm/s)	Layer thickness (μm)	Hatch distance (mm)	Preheated build platform ($^{\circ}\text{C}$)	Preheated powder	Protective gas
PH1	200	100	1000	20	0.1	80	No	Nitrogen
316L	200	100	1000	20	0.1	80	No	Argon

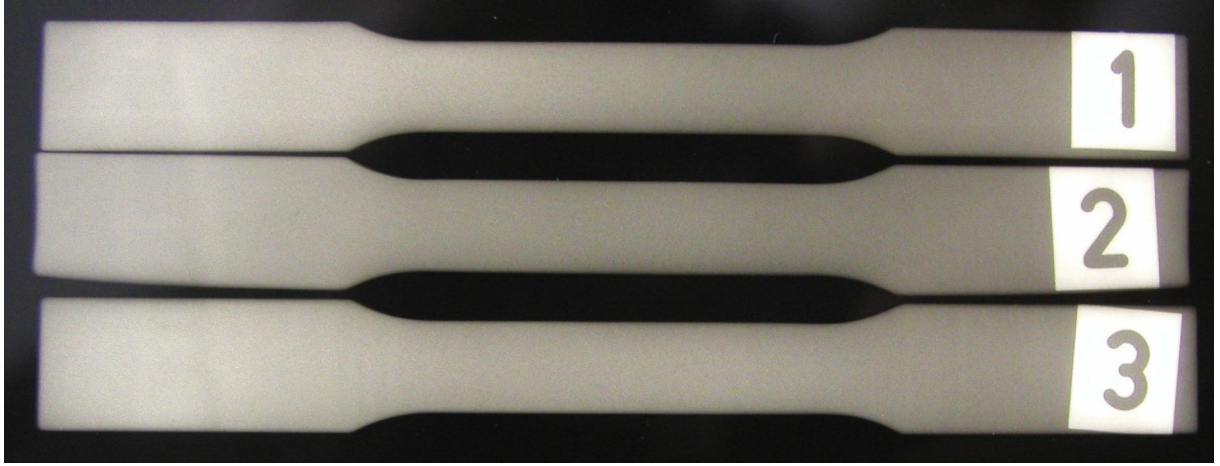


Figure 6.6: An X-ray image of the PH1-1, PH1-2 and PH1-3 tensile coupons.

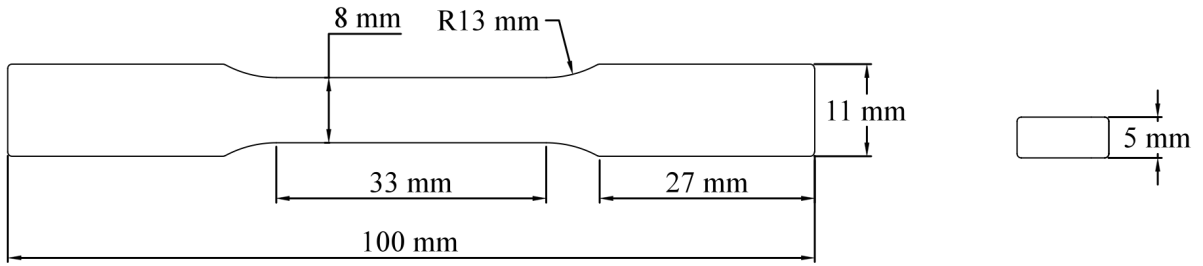


Figure 6.7: The nominal dimensions of the PH1 and 316L tensile coupons.

using PBF with both PH1 and 316L powder (leading to 28 coupons in total). One coupon for each orientation built from PH1 and 316L stainless steel powder was tested. The coupons had the traditional dog bone shape with the nominal dimensions shown in Figure 6.7.

The building orientation angles of the tensile coupons are defined in Figure 6.8, similar to Luecke and Slotwinski (2014), where θ is the angle measured from the horizontal building plane to the longitudinal axis of the coupon (i.e. $\theta = 0^\circ$ is a coupon parallel to the building plane and $\theta = 90^\circ$ is a vertical coupon), ψ is the angle between the recoating direction and the longitudinal axis of the coupon (i.e. $\psi = 0^\circ$ is aligned with the recoating direction and $\psi = 90^\circ$ is perpendicular to the recoating direction) and ϕ is the angle between the long face of the cross-section and the building plane (i.e. $\phi = 0^\circ$ is a coupon with its long face parallel to the building plane and $\phi = 90^\circ$ is a coupon with its short face parallel to the building plane). For the vertical coupons ($\theta = 90^\circ$), $\phi = 0^\circ$ is

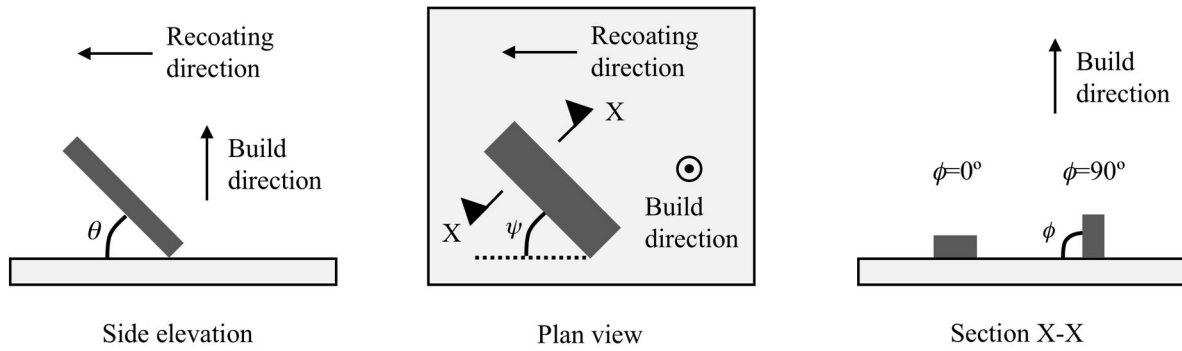


Figure 6.8: Building orientation angles θ , ψ and ϕ for the tensile and compressive coupons.

a coupon with the short face of its cross-section parallel to the recoating direction and $\phi = 90^\circ$ with its long face parallel to the recoating direction.

The tensile tests were carried out at room temperature and in compliance with EN ISO 6892-1 (2009) using an Instron 8802 testing machine, with the data being recorded with a proprietary in-house data logger at 2 Hz frequency. The coupons were measured prior to testing and had the standard gauge lengths ($5.65\sqrt{S_0}$, where S_0 is the original cross-sectional area of the parallel length) marked upon their surface. The instrumentation consisted of two electrical resistance strain gauges, attached to the side faces of the coupons at mid-height to measure strains until shortly after material yielding and a video extensometer to monitor the position of two dots painted onto the front face of the coupon to measure post yielding strains, as shown in Figure 6.9. The applied tensile load was measured directly from a load cell in the testing machine. As recommended in EN ISO 6892-1 (2009), two crosshead separation rates were used before and post material yielding, dependent upon the parallel length of the necked region, with a gradual ramp between the two rates. The PH1-9 coupon was tested first and fractured immediately following the programmed ramp; as a precaution only the slower rate was used for the remaining PH1 coupons. The low failure strain and sudden fracture is an inherent property of some PBF metallic materials. The two separation rates with a gradual ramp were used for the 316L coupons.



Figure 6.9: A fractured 316L tensile coupon at the end of a test.

The Young's modulus E , 0.2% proof stress $\sigma_{0.2}$, 1.0% proof stress $\sigma_{1.0}$, ultimate tensile stress σ_u , strain at the ultimate tensile stress ε_u , fracture strain measured over the marked standard gauge length ε_f , area reduction at fracture A_f , the Ramberg–Osgood (R–O) parameter n (Ramberg and Osgood, 1943) and the extended R–O parameters $n'_{0.2,1.0}$ and $n'_{0.2,u}$ (Mirambell and Real, 2000; Rasmussen, 2003; Gardner and Ashraf, 2006) determined from the coupon tests are reported in Tables 6.3 and 6.4 for the PH1 and 316L coupons respectively. The average material properties are also reported for the horizontal ($\theta = 0^\circ$), vertical ($\theta = 90^\circ$) and inclined ($\theta = 45^\circ$) coupons; average values for all orientations are also presented. Typical material properties from the manufacturer's datasheets (EOS, 2015, 2014) are provided in Table 6.5, which, it should be noted, are not guaranteed minimum values (Luecke and Slotwinski, 2014). The stress–strain curves for all the tested tensile coupons are shown in Figures 6.10 and 6.11. The variation of Young's modulus E , 0.2% proof stress $\sigma_{0.2}$, ultimate tensile stress σ_u and fracture strain ε_f with coupon building orientation θ is plotted in Figures 6.12 to 6.15 for the PH1 coupons, together with existing heat-treated PH1 tensile coupon data (Khalid Rafi et al., 2013), and Figures 6.16 to 6.19 for the 316L coupons, alongside existing 316L tensile coupon results (Tolosa et al., 2010; Mertens et al., 2014; Spierings et al., 2011; Mower and Long, 2016; Meier and Haberland, 2008). The plotted tensile material properties have been normalised by the average of the respective dataset. Note that PH1-14 was a replacement coupon that was built several months later than the original PH1 coupons and exhibits a significantly lower Young's modulus E and 0.2% proof stress $\sigma_{0.2}$ compared with the other inclined coupons, and as a result has been omitted from Figures 6.12 to 6.15.

The tested PH1 tensile coupons show fairly consistent values of the Young's modulus E , varying $\pm 3\%$, with a very slight reduction as the building angle θ increases as seen in Figure 6.12. Likewise, the 0.2% proof stress $\sigma_{0.2}$ shows minimal variation with the building angle (see Figure 6.13), while the existing heat-treated data (Khalid Rafi et al., 2013) has a significantly lower yield stress for the vertical ($\theta = 90^\circ$) coupon. There is a clear trend with both the ultimate tensile stress σ_u and fracture strain ε_f of decreasing

Table 6.3: Summary of the PH1 measured tensile coupon material properties.

Coupon	θ (deg)	ψ (deg)	ϕ (deg)	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ε_u (%)	ε_f (%)	A_f (%)	n	$n'_{0.2,1.0}$	$n'_{0.2,u}$
PH1-1	0	0	0	194900	870	1045	1273	10.0	13.4	37.5	7.5	2.5	2.6
PH1-2	0	0	90	191100	816	1022	1252	7.2	14.1	39.8	8.3	2.5	2.6
PH1-3	0	90	0	197300	883	1062	1280	8.3	15.1	41.5	7.3	2.5	2.5
PH1-4	0	90	90	194600	815	1030	1262	8.1	16.4	40.8	8.1	2.5	3.4
PH1-5	0	45	0	192800	879	1055	1255	7.8	15.3	35.9	6.5	2.4	2.5
PH1-6	0	45	90	189500	826	1031	1248	7.8	15.7	45.1	8.4	2.5	2.7
PH1-7	90	-	0	188700	850	1111	1216	4.6	5.4	7.2	5.1	2.4	2.6
PH1-8	90	-	90	187700	858	1113	1178	2.6	4.3	10.8	5.3	1.8	2.5
PH1-9	45	0	0	187500	800	1072	1232	4.7	3.6	9.9	5.0	2.5	3.4
PH1-10	45	0	90	193000	822	1074	1203	3.4	4.6	13.0	5.6	2.5	2.5
PH1-11	45	90	0	188400	810	1074	1238	6.5	12.5	30.5	5.4	2.5	3.5
PH1-12	45	90	90	198200	819	1078	1234	4.9	11.1	46.4	5.3	2.5	2.6
PH1-13	45	45	0	196900	803	1084	1257	6.7	11.4	33.0	4.9	2.5	3.5
PH1-14	45	45	90	180600	710	1012	1245	7.2	7.7	14.6	6.1	3.5	4.6
Average (horizontal)	0	-	-	193400	848	1041	1262	8.2	15.0	40.1	7.7	2.5	2.7
Average (vertical)	90	-	-	188200	854	1112	1197	3.6	4.9	9.0	5.2	2.1	2.5
Average (inclined)	45	-	-	190800	794	1066	1235	5.6	8.5	24.6	5.4	2.7	3.4
Average (all)	-	-	-	191500	826	1062	1241	6.4	10.8	29.0	6.3	2.5	3.0

Table 6.4: Summary of the 316L measured tensile coupon material properties.

Coupon	θ (deg)	ψ (deg)	ϕ (deg)	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ε_u (%)	ε_f (%)	A_f (%)	n	$n'_{0.2,1.0}$	$n'_{0.2,u}$
316L-15	0	0	0	179200	486	530	626	24.3	28.9	33.6	10.7	1.6	2.5
316L-16	0	0	90	171900	492	533	637	27.0	37.7	35.6	13.5	1.8	2.5
316L-17	0	90	0	167300	480	510	616	23.9	29.5	25.6	14.0	1.6	1.9
316L-18	0	90	90	167500	488	529	634	27.2	34.7	40.2	12.6	1.8	2.5
316L-19	0	45	0	172000	511	551	659	30.8	41.0	38.3	10.4	1.8	2.5
316L-20	0	45	90	170500	499	542	645	27.4	38.7	46.7	11.6	1.6	2.5
316L-21	90	-	0	183700	391	469	552	24.5	28.0	34.2	2.5	2.5	3.6
316L-22	90	-	90	180300	428	488	576	37.2	50.3	67.7	3.3	2.4	3.5
316L-23	45	0	0	177800	502	538	658	46.6	59.5	53.8	5.5	2.4	2.5
316L-24	45	0	90	180000	466	526	637	40.9	51.5	55.3	3.5	2.5	3.3
316L-25	45	90	0	180900	468	513	631	44.8	57.8	44.6	4.2	2.4	2.7
316L-26	45	90	90	202900	486	536	660	42.6	56.6	49.1	4.4	2.5	2.7
316L-27	45	45	0	195600	477	524	645	44.2	56.5	48.9	3.4	2.4	2.7
316L-28	45	45	90	179100	482	524	649	44.2	52.9	45.2	4.8	2.4	2.6
Average (horizontal)	0	-	-	171400	493	533	636	26.8	35.1	36.7	12.1	1.7	2.4
Average (vertical)	90	-	-	182000	409	478	564	30.9	39.2	51.0	2.9	2.5	3.5
Average (inclined)	45	-	-	186100	480	527	647	43.9	55.8	49.5	4.3	2.4	2.7
Average (all)	-	-	-	179200	475	522	630	34.7	44.5	44.2	7.5	2.1	2.7

Table 6.5: Summary of typical PH1 and 316L material properties.

Material	Source	θ (deg)	ψ (deg)	ϕ (deg)	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	σ_u (N/mm ²)	ε_f (%)
PH1 ^a	Datasheet (EOS, 2015)	0	0-90	-	- ^b	950-1100	1150-1250	13-21
PH1 ^a	Datasheet (EOS, 2015)	90	-	0-90	- ^b	865-1015	1150-1250	10-18
316L ^a	Datasheet (EOS, 2014)	0	0-90	-	185000	470-590	590-690	25-55
316L ^a	Datasheet (EOS, 2014)	90	-	0-90	180000	380-560	485-595	30-70
316L	EN 1993-1-4 (2015)	-	-	-	200000	200-240	500-530	- ^b
316L	EN 10088-2 (2014b)	-	-	-	- ^b	220-240	520-680	40-45

^aAs manufactured without heat treatment

^bNot provided

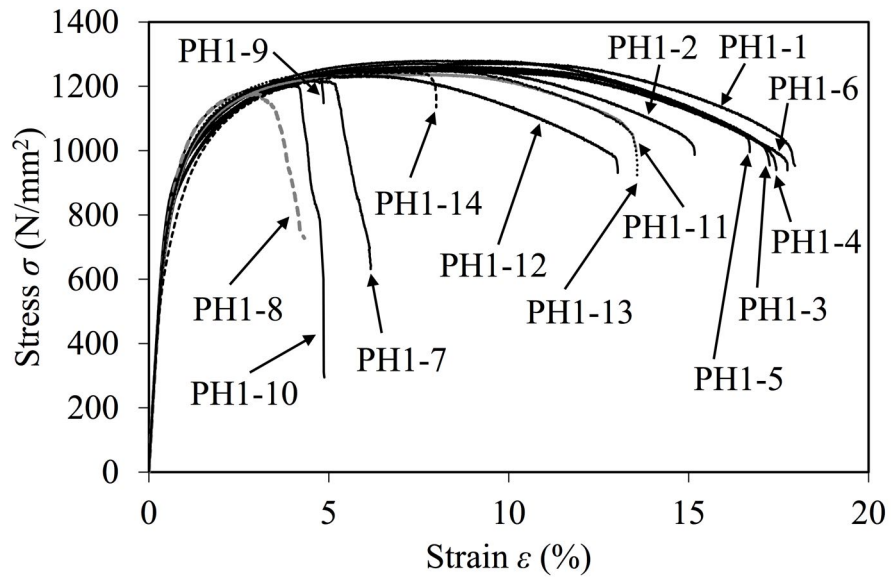


Figure 6.10: Measured stress–strain curves from the tensile PH1 coupons.

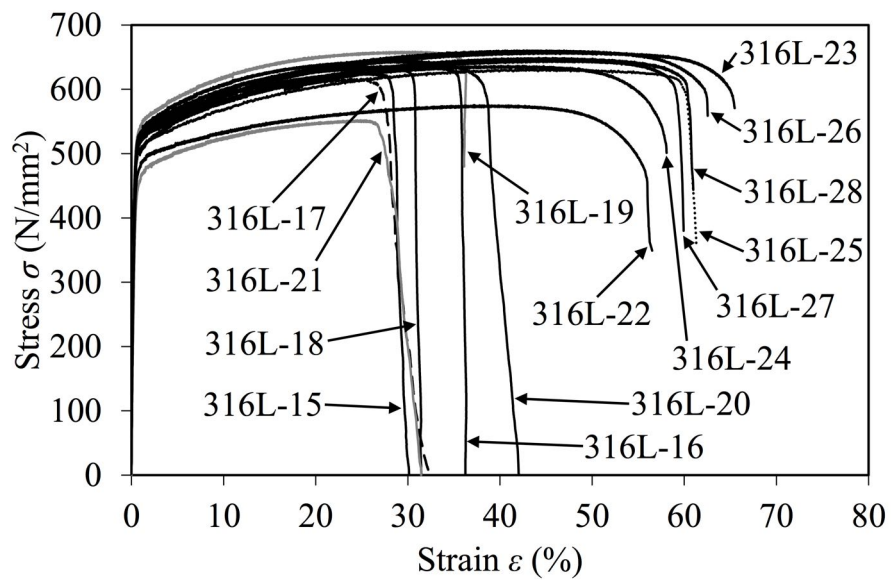


Figure 6.11: Measured stress–strain curves from the tensile 316L coupons.

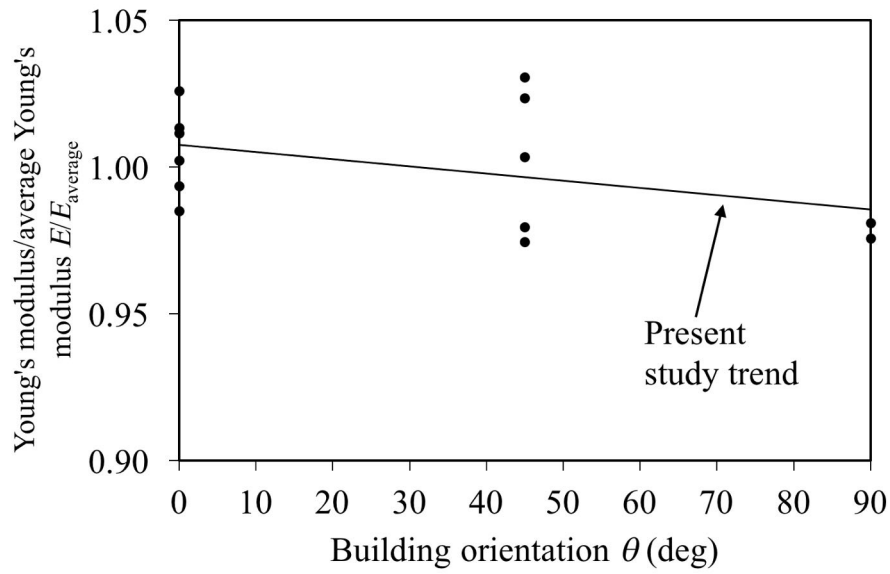


Figure 6.12: Normalised Young's moduli varying with building orientation for the tensile PH1 coupons.

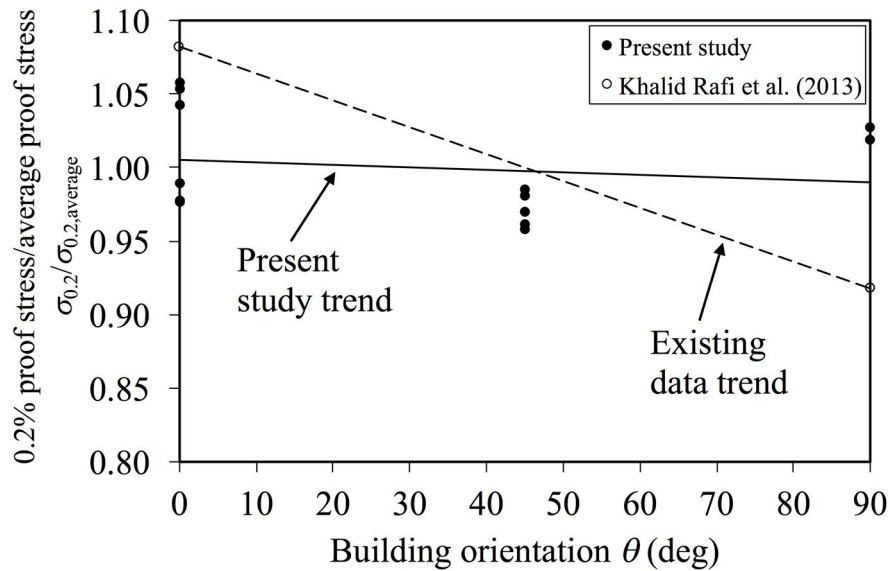


Figure 6.13: Normalised 0.2% proof stress varying with building orientation for the tensile PH1 coupons.

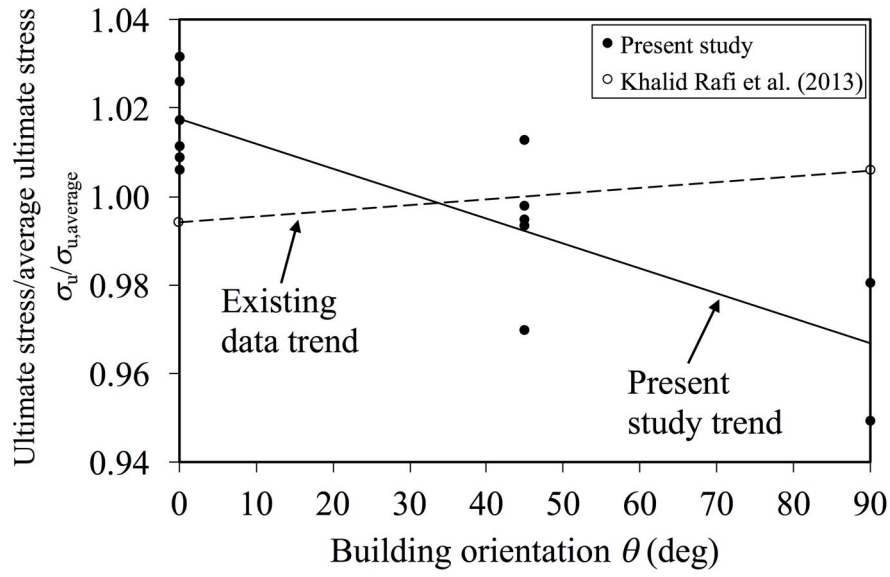


Figure 6.14: Normalised ultimate stress varying with building orientation for the tensile PH1 coupons.

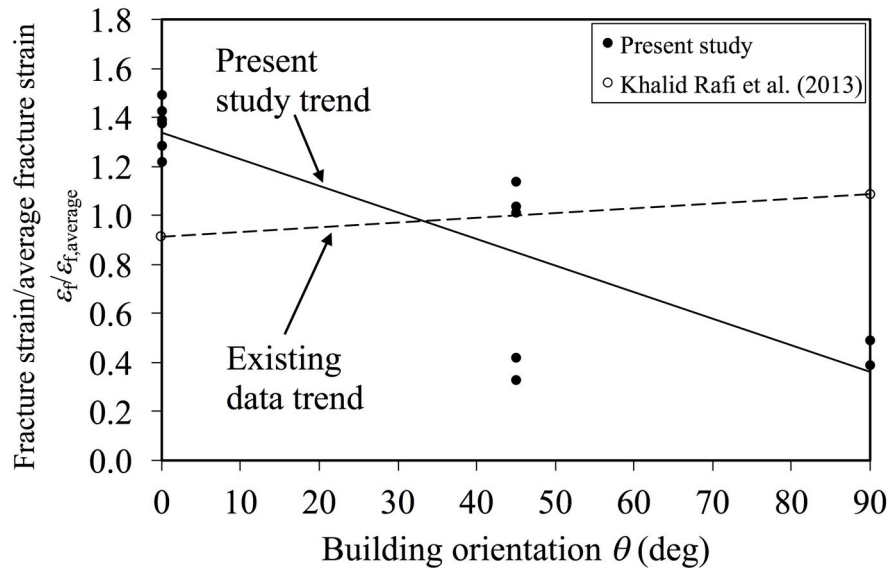


Figure 6.15: Normalised fracture strain varying with building orientation for the tensile PH1 coupons.

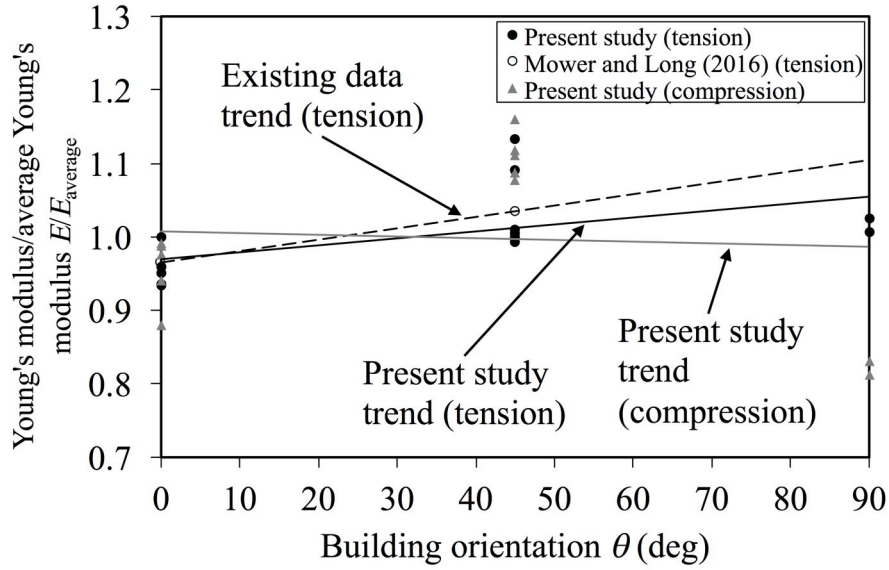


Figure 6.16: Normalised Young's moduli varying with building orientation for the tensile 316L coupons.

magnitude with increasing building angle (see Figures 6.14 and 6.15 respectively), the latter quite dramatically, particularly for the vertical coupons ($\theta = 90^\circ$) and those inclined in the recoating direction ($\theta = 45^\circ$, $\psi = 0^\circ$), which exhibited negligible necking before failure. For the existing data (Khalid Rafi et al., 2013) there is no clear influence of building orientation on the ultimate tensile stress σ_u and fracture strain ε_f indicating that heat treatment reduces the inherent material anisotropy. The angle of the failure surfaces on the tested vertical ($\theta = 90^\circ$) and inclined ($\theta = 45^\circ$) PH1 coupons were clearly aligned with the building plane, indicating that the building plane is a weaker plane. It should be noted that Khalid Rafi et al. (2013) carried out the tensile coupon tests in accordance with ASTM E8M (2016), which utilises a different gauge length to EN ISO 6892-1 (2009).

The Young's moduli E for the 316L tensile coupons show minimal variation with building angle θ , which is in line with previous stress relieved coupon results (Mower and Long, 2016), as can be seen from Figure 6.16. For the 0.2% proof stress $\sigma_{0.2}$ and ultimate tensile stress σ_u there is a clear reduction in magnitude with a greater building orientation, as seen in Figures 6.17 and 6.18 respectively, which is also apparent from the existing 316L tension coupon tests from the literature (Tolosa et al., 2010; Mertens et al., 2014;

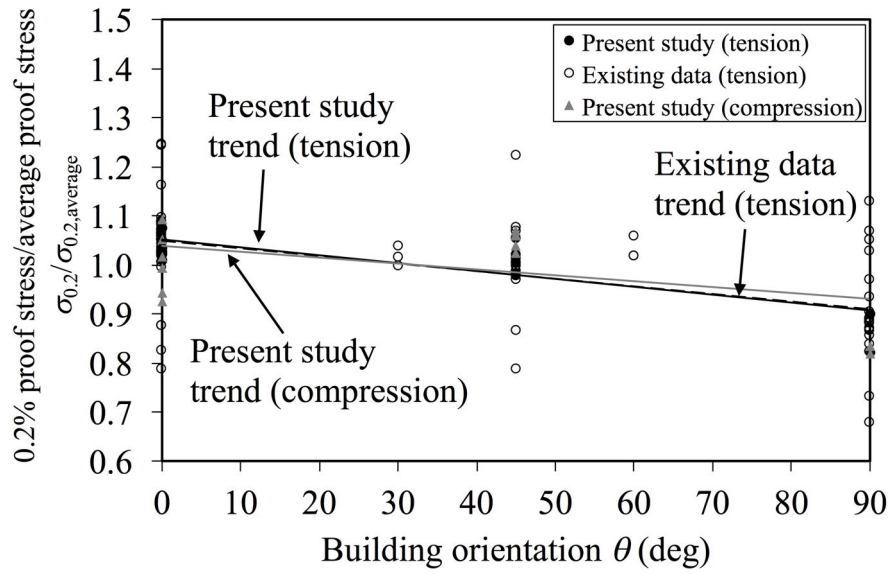


Figure 6.17: Normalised 0.2% proof stress varying with building orientation for the tensile 316L coupons.

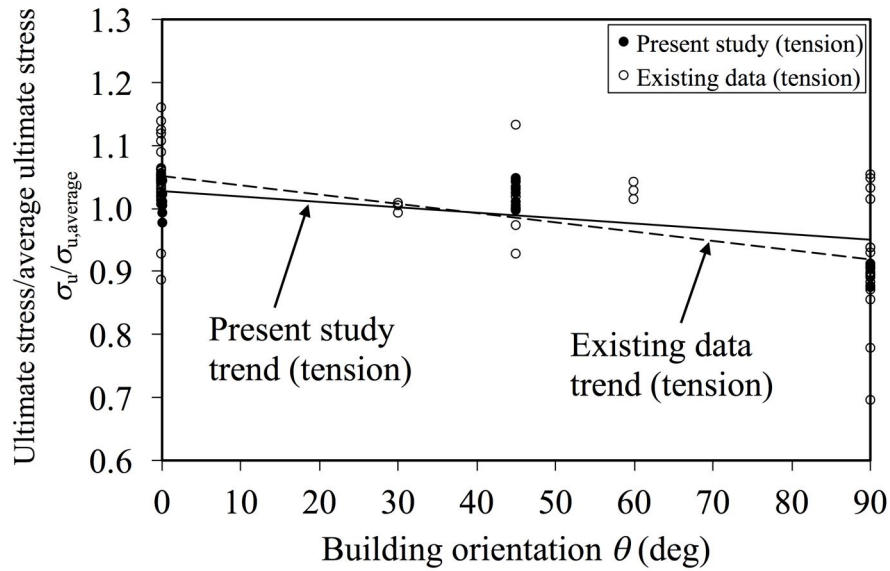


Figure 6.18: Normalised ultimate stress varying with building orientation for the tensile 316L coupons.

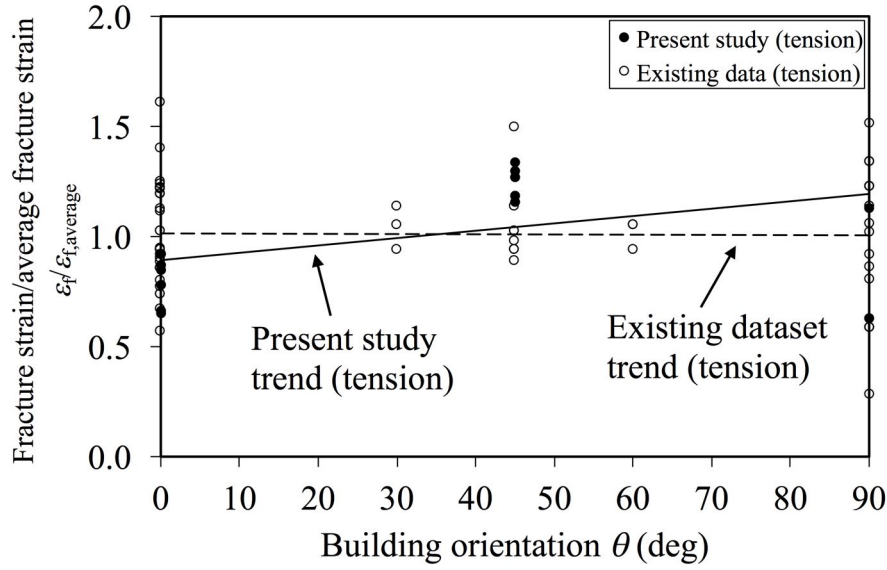


Figure 6.19: Normalised fracture strain varying with building orientation for the tensile 316L coupons.

Spierings et al., 2011; Mower and Long, 2016; Meier and Haberland, 2008); this is attributed to elongated grains in the vertical direction leading to a lower yield stress when loaded in this orientation (Mertens et al., 2014). In both Figures 6.17 and 6.18 the linear regression trend line for the present study is very close to that from the literature. There is substantial variation in fracture strain ε_f , but no clear trend with building orientation from both the generated and existing data sets (Tolosa et al., 2010; Mertens et al., 2014; Spierings et al., 2011; Mower and Long, 2016; Meier and Haberland, 2008), as seen in Figure 6.19. Meier and Haberland (2008), Spierings et al. (2011) and Mertens et al. (2014) observed their vertical coupons fracturing at the lowest strain of all orientations, as low as 8% due to ‘lack of melting’ defects that were observed on the fracture surface; in comparison, the two 316L vertical coupons tested in this study reached 28% and 50% strains at fracture and Tolosa et al. (2010) noted that the vertical coupons exhibited the largest fracture strains (42–44% on average). It should however be highlighted that only two of the existing studies (Mertens et al., 2014; Spierings et al., 2011) state that they undertook their tensile coupon tests in accordance with EN ISO 6892-1 (2009), while the other studies (Tolosa et al., 2010; Mower and Long, 2016; Meier and Haberland, 2008) make no mention of adhering to a particular standard, and hence may have used

different gauge lengths in their calculation of the fracture strain ε_f .

An observation during testing was that the less ductile 316L coupons (i.e. the horizontally orientated coupons ($\theta = 0^\circ$) and one vertical coupon (316L-21)), did not fracture cleanly, but instead slowly and silently tore apart, whereas the remaining coupons all fractured with a characteristic ‘bang’. Additionally, unlike for the PH1 coupons, the 316L tensile coupon failure surfaces did not obviously align with the building plane. It is also unclear whether heat treatment can remediate the anisotropy observed in PBF 316L tensile coupons. The powder datasheet (EOS, 2014) refers to stress relieving heat treatment, but does not refer to changes to the mechanical properties. Mower and Long (2016) tested horizontal ($\theta = 0^\circ$) and inclined ($\theta = 45^\circ$) thermally stress relieved 316L coupons and observed similar properties, but did not test vertically ($\theta = 90^\circ$) built coupons or non heat-treated coupons. Yadollahi et al. (2015) tested heat treated DED 316L stainless steel tensile coupons and observed effects similar to annealing, with an increased and more consistent fracture strain and reduced yield and ultimate strengths.

Comparison can also be made with typical horizontal ($\theta = 0^\circ$) and vertical ($\theta = 90^\circ$) mechanical properties from the powder datasheets (EOS, 2015, 2014) given in Table 6.5. For the PH1 coupons, the yield stress was lower than expected, while the ultimate stresses and fracture strain for $\theta = 0^\circ$ were generally within the stated range, although the fracture strain for $\theta = 90^\circ$ was significantly below that expected. By contrast the 316L material properties were all within the expected range, apart from the horizontal Young’s moduli, which were lower than expected. The PBF 316L properties can also be compared with those of conventionally formed 316L stainless steel from EN 1993-1-4 (2015) and EN 10088-2 (2014b), as presented in Table 6.5. It is apparent from the results that PBF 316L stainless steel has a slightly lower Young’s modulus, but a significantly higher yield strength and more variable fracture strain than conventional hot-rolled 316L material. The increase in strength has also been noted by others (Tolosa et al., 2010; Mower and Long, 2016; Meier and Haberland, 2008) and has been attributed to the very rapid

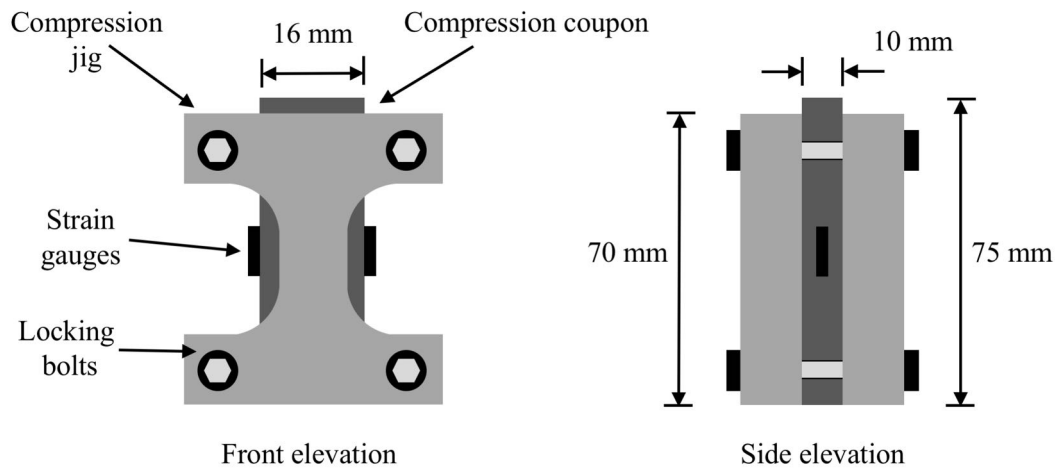


Figure 6.20: The compression coupon setup with the nominal coupon dimensions.

cooling and solidification associated with PBF manufactured material leading to a very fine crystalline structure. This highlights that the mechanical properties are not just dependent upon the composition of the material, but also strongly influenced by the adopted manufacturing processes (Tolosa et al., 2010).

6.2.3 Compressive coupon tests

Compressive coupon tests were carried out to measure the compressive stress–strain response of the material, to determine if the directionality effects also exist in compression and to compare the tensile and compressive properties. Compression coupons were only built with the 316L powder, with fourteen coupons manufactured in the same orientations as those used for the tensile coupons (defined in Figure 6.8), with one coupon tested per building orientation. The coupons were built in a cuboidal shape with constant cross-sectional area, with the nominal dimensions shown in Figure 6.20.

All of the as-built compression coupons had their ends machined to ensure that they were flat and parallel. The side faces were also machined on some coupons to remove the support structures and where the cross-sectional area varied along coupon length.



Figure 6.21: The compression coupon test setup.

An Instron 8802 testing machine was again used, with metallic dowels inserted into the jaws to apply the compressive load to the coupons ends. The geometry of the coupons was measured prior to testing and two electrical resistance strain gauges were affixed at mid-height on opposite faces. The applied load, measured by means of a load cell within the testing machine, and strain gauge data was recorded at 2 Hz frequency. To prevent minor axis buckling the compression coupons were held within a restraining jig, as shown in Figures 6.20 and 6.21 with the sides of the coupon in contact with the jig covered in a thin layer of lubricating grease to reduce friction effects, with the locking bolts finger tight such that buckling was restrained but Poisson's ratio effect expansion could still occur. This type of setup has been used successfully in previous studies (Rockey and Jenkins, 1957; Rasmussen and Hancock, 1993a; Gardner and Nethercot, 2004a; Foster and Gardner, 2013). A constant compression rate of 0.067 mm/min was used, with the test stopped as the metallic dowel was about to touch the top face of the compression jig.

The compressive Young's modulus E_c , 0.2% proof stress $\sigma_{0.2,c}$, 1.0% proof stress $\sigma_{1.0,c}$

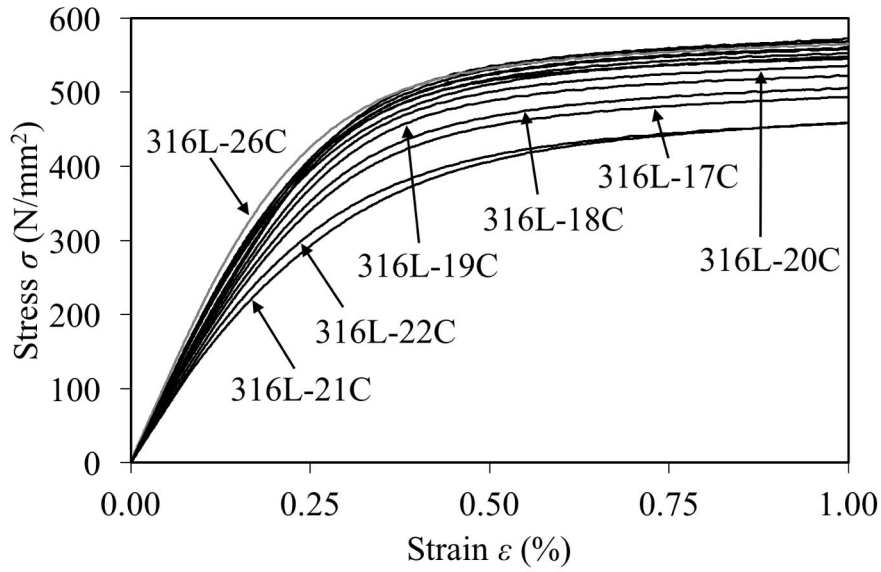


Figure 6.22: Measured stress–strain curves from the compressive 316L coupons.

and Ramberg–Osgood parameter n_c (Ramberg and Osgood, 1943), along with ratios of the compressive to tensile Young’s moduli E_c/E , 0.2% proof stresses $\sigma_{0.2,c}/\sigma_{0.2}$ and 1.0% proof stresses $\sigma_{1.0,c}/\sigma_{1.0}$ are reported in Table 6.6. The average material properties are again also reported for the horizontal ($\theta = 0^\circ$), vertical ($\theta = 90^\circ$) and inclined ($\theta = 45^\circ$) coupons and averaged values for all orientations. The stress–strain curves for all the tested austenitic 316L compressive coupons are shown in Figure 6.22 to provide an indication of the range of stress–strain responses; a sample of these curves have been labelled. The variation of Young’s modulus E_c and 0.2% proof stress $\sigma_{0.2,c}$ with the coupon longitudinal axis building orientation θ is plotted in Figures 6.16 and 6.17 respectively for the compressive coupons along with the new and existing (Tolosa et al., 2010; Mertens et al., 2014; Spierings et al., 2011; Mower and Long, 2016; Meier and Haberland, 2008) 316L tensile coupons. The plotted compressive material properties have been normalised by the average of all the compressive results.

Plotting the compression coupon Young’s moduli E_c against the building angle θ , in Figure 6.16, shows that the relationship between the magnitude and building angle is relatively inconsistent — the inclined coupons ($\theta = 45^\circ$) have the largest Young’s moduli values, while the vertical coupons ($\theta = 90^\circ$) have the lowest values. The value of the

Table 6.6: Summary of the 316L compressive coupon measured material properties.

Coupon	θ (deg)	ψ (deg)	ϕ (deg)	E_c (N/mm ²)	$\sigma_{0.2,c}$ (N/mm ²)	$\sigma_{1.0,c}$ (N/mm ²)	η_c	E_c/E	$\sigma_{0.2,c}/\sigma_{0.2}$	$\sigma_{1.0,c}/\sigma_{1.0}$
316L-15C	0	0	0	181200	512	553	8.5	1.01	1.05	1.04
316L-16C	0	0	90	181900	533	599	8.5	1.06	1.08	1.12
316L-17C	0	90	0	161400	450	504	6.3	0.96	0.94	0.99
316L-18C	0	90	90	172500	459	515	5.8	1.03	0.94	0.97
316L-19C	0	45	0	172700	484	530	7.2	1.00	0.95	0.96
316L-20C	0	45	90	179100	496	545	7.1	1.05	0.99	1.01
316L-21C	90	-	0	149100	398	472	4.3	0.81	1.02	1.01
316L-22C	90	-	90	152600	407	472	5.4	0.85	0.95	0.97
316L-23C	45	0	0	197800	517	565	6.3	1.11	1.03	1.05
316L-24C	45	0	90	199500	499	555	5.8	1.11	1.07	1.06
316L-25C	45	90	0	205000	521	580	5.9	1.13	1.11	1.13
316L-26C	45	90	90	212800	522	574	7.1	1.05	1.07	1.07
316L-27C	45	45	0	199500	516	566	6.1	1.02	1.08	1.08
316L-28C	45	45	90	203800	506	560	6.4	1.14	1.05	1.07
Average (horizontal)	0	-	-	174800	489	541	7.2	1.02	0.99	1.02
Average (vertical)	90	-	-	150800	403	472	4.8	0.83	0.99	0.99
Average (inclined)	45	-	-	203100	513	567	6.2	1.09	1.07	1.08
Average (all)	-	-	-	183500	487	542	6.5	1.02	1.02	1.04

0.2% proof stress $\sigma_{0.2,c}$ typically decreases with increasing building angle θ , similar to the behaviour and magnitudes seen in tension, as apparent from Figure 6.17. As observed for the Young's modulus, on average the inclined coupons ($\theta = 45^\circ$) have the largest 0.2% proof stress $\sigma_{0.2,c}$ of all the studied building orientations (see Table 6.6).

Gardner and Nethercot (2004a) compared the material behaviour of conventionally produced tensile and compressive coupons and noted that on average (considering 54 tensile and 56 compressive results) the Young's modulus E and 1.0% proof stress $\sigma_{1.0}$ were 1% and 4% higher respectively for the tested compressive coupons than the corresponding tensile coupons, while the 0.2% proof stress $\sigma_{0.2}$ was 5% lower on average in compression than in tension. The additive manufactured 316L austenitic stainless steel coupons tested here also generally show slightly non-symmetric stress–strain responses on average. The Young's modulus E and 0.2% proof stress $\sigma_{0.2}$ were both observed to be 2% higher on average in compression than tension, while the 1.0% proof stress $\sigma_{1.0}$ was found to be 4% higher on average in compression. The only major deviation amongst the building orientation averages is that for the vertical coupons the average tensile Young's modulus is greater than the equivalent compression values, as seen in Table 6.6. Gardner and Nethercot (2004a) also observed that the compressive coupons tended to have a more rounded stress–strain response than their tensile equivalents, with a lower Ramberg–Osgood parameter n (Afshan et al., 2013), which can also be observed here from the averaged values in Tables 6.4 and 6.6. It is apparent from Table 6.6 that while the ratio of the compressive property to the tensile property is reasonably consistent for the Young's modulus E and 1.0% proof stress $\sigma_{1.0}$, there is more variation for the 0.2% proof stress $\sigma_{0.2}$.

Comparison with the manufacturer's powder datasheet (EOS, 2014) in Table 6.5 shows that the horizontal compression coupons ($\theta = 0^\circ$) have 0.2% proof stresses $\sigma_{0.2}$ that typically lie within the expected range, although two results are slightly lower, while the vertical coupon ($\theta = 90^\circ$) results all lie within the stated range. For both building

orientations, the Young's modulus E is lower than that expected from the datasheet. Comparison with the stated Young's moduli for conventionally formed 316L stainless steel from EN 1993-1-4 (2015) and EN 10088-2 (2014b) shows that, similar to the tension observations, the measured Young's moduli are lower than typically expected for the conventionally produced material, though the 0.2% proof stresses $\sigma_{0.2}$ are higher than the quoted minimum specified values.

6.2.4 Stub column geometric properties

The stub columns had nominal 50×50 mm outer dimensions with wall thicknesses varying from 1 mm to 5 mm in order to examine a wide range of local cross-section slendernesses. The geometric properties of the stub column specimens were carefully measured before testing, following the recommendations laid out in EN 10219 (2006a). The outer breadth B and height H were measured across the four faces at five equally spaced locations along the column length using digital callipers. The average wall thickness t was calculated from the wall thicknesses measured at three locations along each face at both ends of the stub columns. The average column length L could similarly be determined by measuring the length of each face of the stub column, again using callipers. The average inner and outer corner radii, r_i and r_e respectively, were measured at both ends by scanning the specimen ends, with an object of known size, and fitting circles using AutoCAD (Autodesk, 2015a); EN 10219 (2006a) suggests using a radius gauge but due to the small radii and discrete gauges this was found to be less accurate.

The local geometric imperfections of the stub columns were measured prior to testing. A potentiometer was used to measure the surface profile while the stub column moved on a milling bed, with the horizontal position recorded with a string potentiometer, as shown in Figure 6.23, similar to previous studies (Schafer and Peköz, 1998). Measurements were taken along three equally spaced longitudinal lines on each face, with the maximum local imperfection ω_0 defined as the maximum deviation from a linear regression line

fitted to the measured surface profile. Imperfection profiles for the $50 \times 50 \times 1$ and $50 \times 50 \times 5$ SHS specimens are shown in Figures 6.24 and 6.25. The average geometric properties, along with the nominal inner and outer corner radii, $r_{i,nom}$ and $r_{e,nom}$, and calculated cross-sectional area A of the SHS stub columns are summarised in Table 6.7.

6.2.5 Stub column testing

Stub column tests were undertaken to determine the load carrying capacity and general cross-sectional response under pure compression. The stub columns had a 200 mm nominal length which is sufficiently short to prevent global buckling, but expected to be long enough to contain a representative pattern of residual stress and imperfections (Ziemian, 2010). The testing setup is shown in Figure 6.26. The end plates provide fixed-end conditions, and although the column ends were machined to be flat and parallel, grout was used above the top plate to ensure full surface contact. Instron 600kN-J3D-SPL and Instron 3500kN-SPL testing machines were used for the 1 mm to 4 mm thick specimens and 5 mm thick specimen respectively. The instrumentation consisted of four electrical resistance strain gauges attached at mid-height, at a distance of $3t$ from the edges of two opposite faces. Four potentiometers recorded the vertical movement of the top platen (Liu and Young, 2003; Theofanous and Gardner, 2009; Yuan et al., 2014; Zhao et al., 2015e,c) and the applied load was measured using a load cell within the testing machine. Data were again recorded using a proprietary in-house developed datalogger and software at a frequency of 2 Hz. The tests were displacement controlled, with the top platen moving downwards at a displacement rate of 0.1 mm/min, achieving a similar compressive strain rate as other stainless steel SHS stub columns tests (Liu and Young, 2003; Zhao et al., 2015e,c; Young and Lui, 2005; Zhao et al., 2016b), with the tests continued beyond the peak load to observe the initial unloading behaviour.

Strain visualisation grids were painted onto three faces of the stub columns, although there were issues with the paint bleeding due to the high porosity of the surface (as

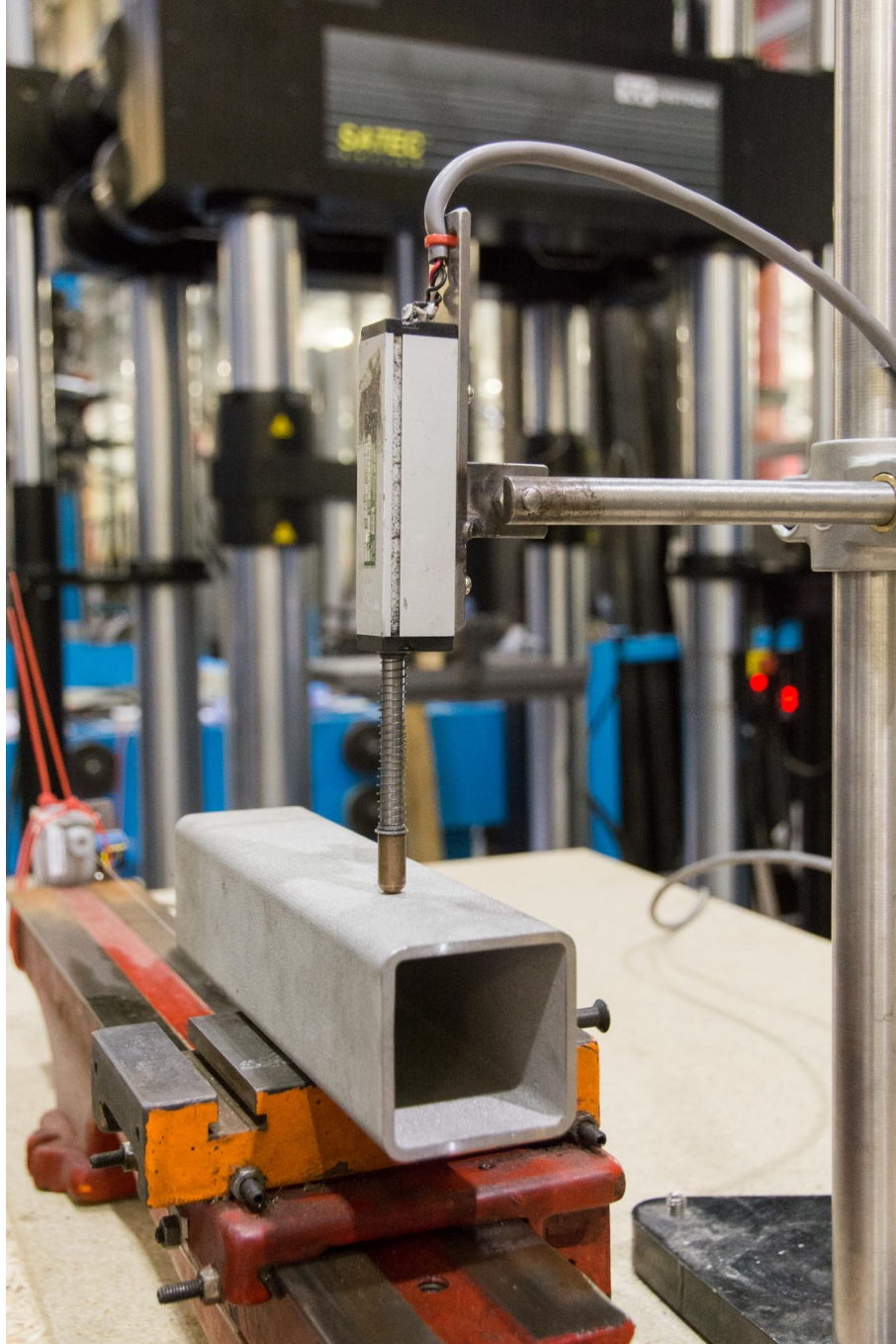


Figure 6.23: The local imperfection measurement setup.

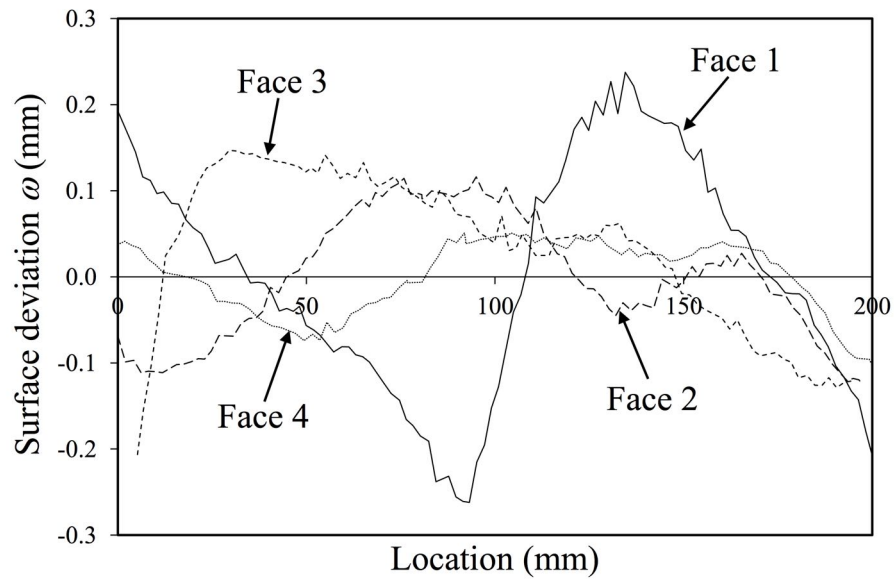


Figure 6.24: Longitudinal imperfection profiles for 50×50×1 SHS specimen.

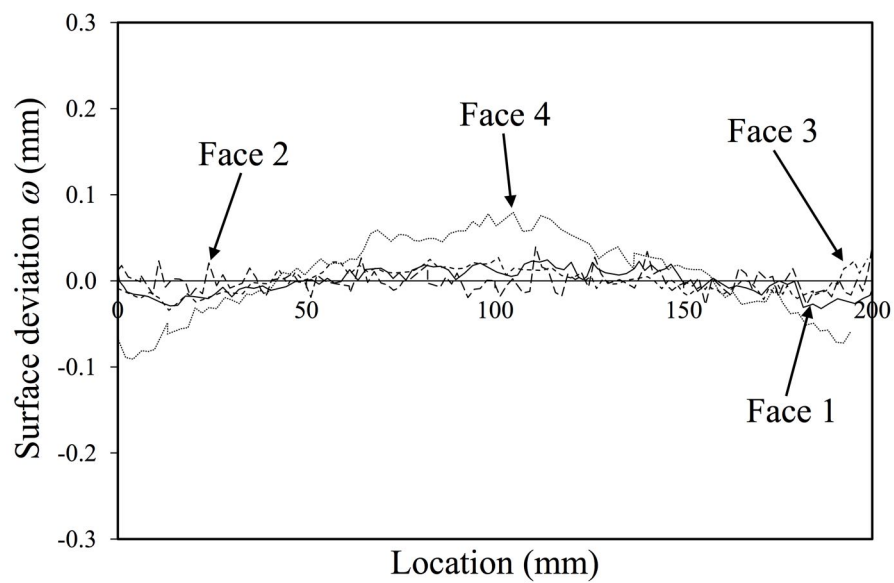


Figure 6.25: Longitudinal imperfection profiles for 50×50×5 SHS specimen.

Table 6.7: Summary of the mean geometric properties of the stub columns.

Specimen	B (mm)	H (mm)	t (mm)	L (mm)	$r_{e,nom}$ (mm)	r_e (mm)	$r_{i,nom}$ (mm)	r_i (mm)	ω_0 (mm)	A (mm ²)
50×50×1	50.17	50.00	1.05	204.34	1.6	1.32	0.6	0.65	0.26	204.1
50×50×2	50.03	50.02	2.03	204.22	3.6	3.10	1.6	1.26	0.11	381.8
50×50×3	49.99	50.03	3.05	204.33	6.0	5.56	3.0	2.60	0.13	549.9
50×50×4	49.99	50.00	4.03	203.76	8.8	8.39	4.8	4.32	0.05	697.5
50×50×5	50.03	50.03	5.02	204.43	12.0	11.52	7.0	6.72	0.09	816.7

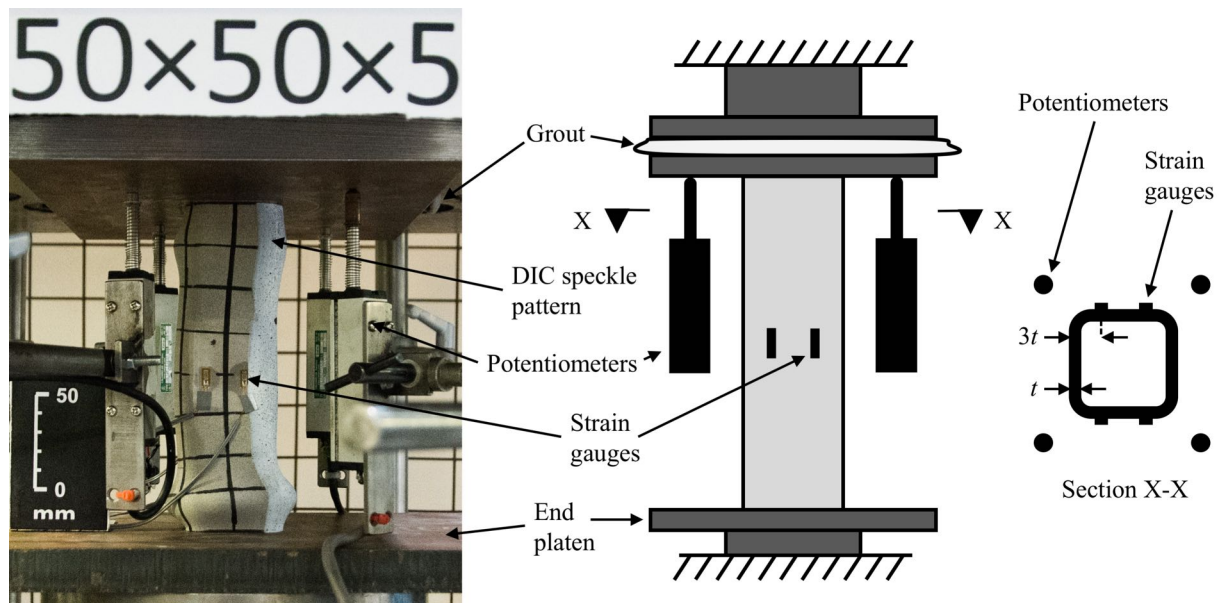


Figure 6.26: The stub column testing setup.

apparent in Figure 6.30) and the inherent surface roughness (visible in Figure 6.27). StrainMaster digital image correlation (DIC) hardware, shown in Figure 6.28, was also used to record the deformations in three-dimensional space by monitoring a speckle pattern that was applied to the fourth face and these were converted into strains and used to verify the initial strain gauge readings.

The ultimate axial load N_u and corresponding true end shortening at the ultimate load δ_u , accounting for the deformation of the end platens, discussed previously in Section 3.4.6, are reported in Table 6.8 and the load–end shortening curves and deformed specimens are presented in Figures 6.29 and 6.30 respectively. The curves show that the $50 \times 50 \times 3$, $50 \times 50 \times 4$ and $50 \times 50 \times 5$ SHS stub columns undergo significant strain hardening before reaching their peak load. The failure mode of the $50 \times 50 \times 1$ SHS column was the classic ‘in-out’ local buckling at mid-height; a similar mode was observed for the $50 \times 50 \times 2$ and $50 \times 50 \times 3$ SHS specimens, but close to one end. The $50 \times 50 \times 4$ and $50 \times 50 \times 5$ SHS columns developed an ‘elephant’s foot’ type local buckling close to both ends, followed by global buckling beyond the peak load, as shown in Figures 6.26 and 6.30.

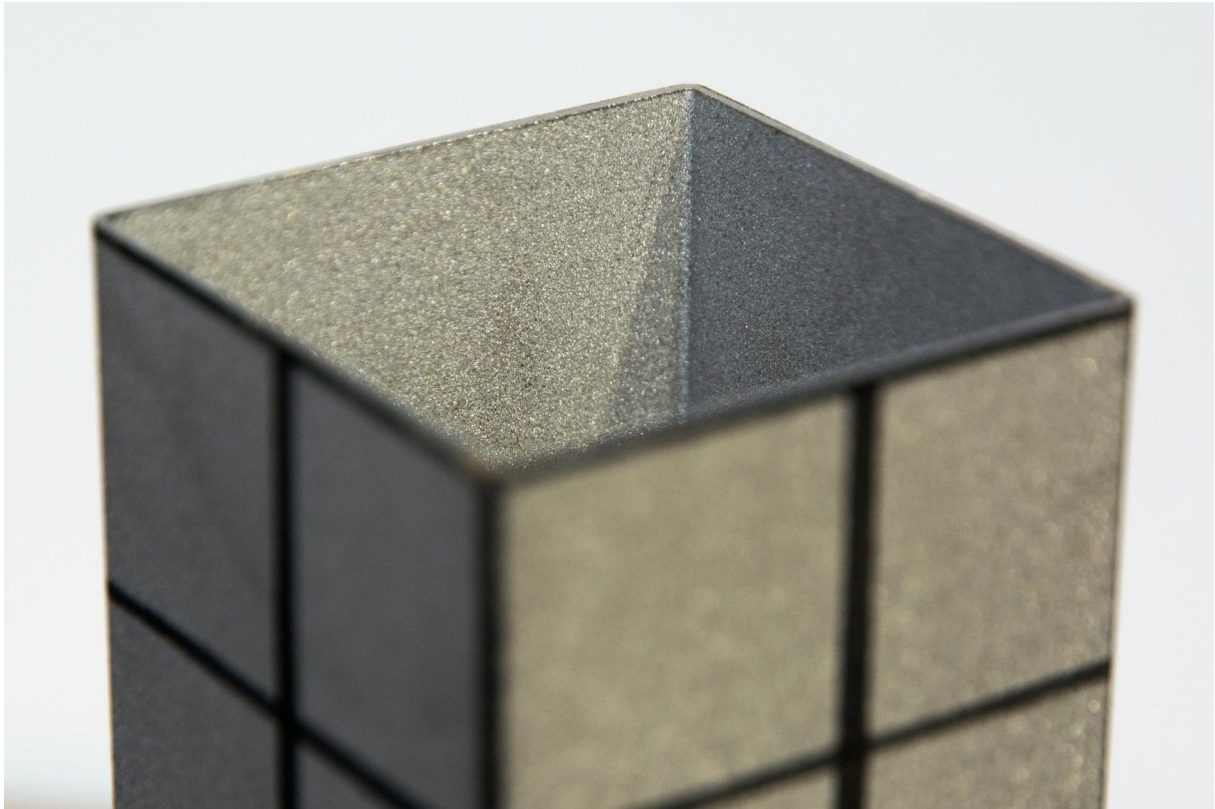


Figure 6.27: Unprocessed rough surface finish due to the novel manufacturing process.

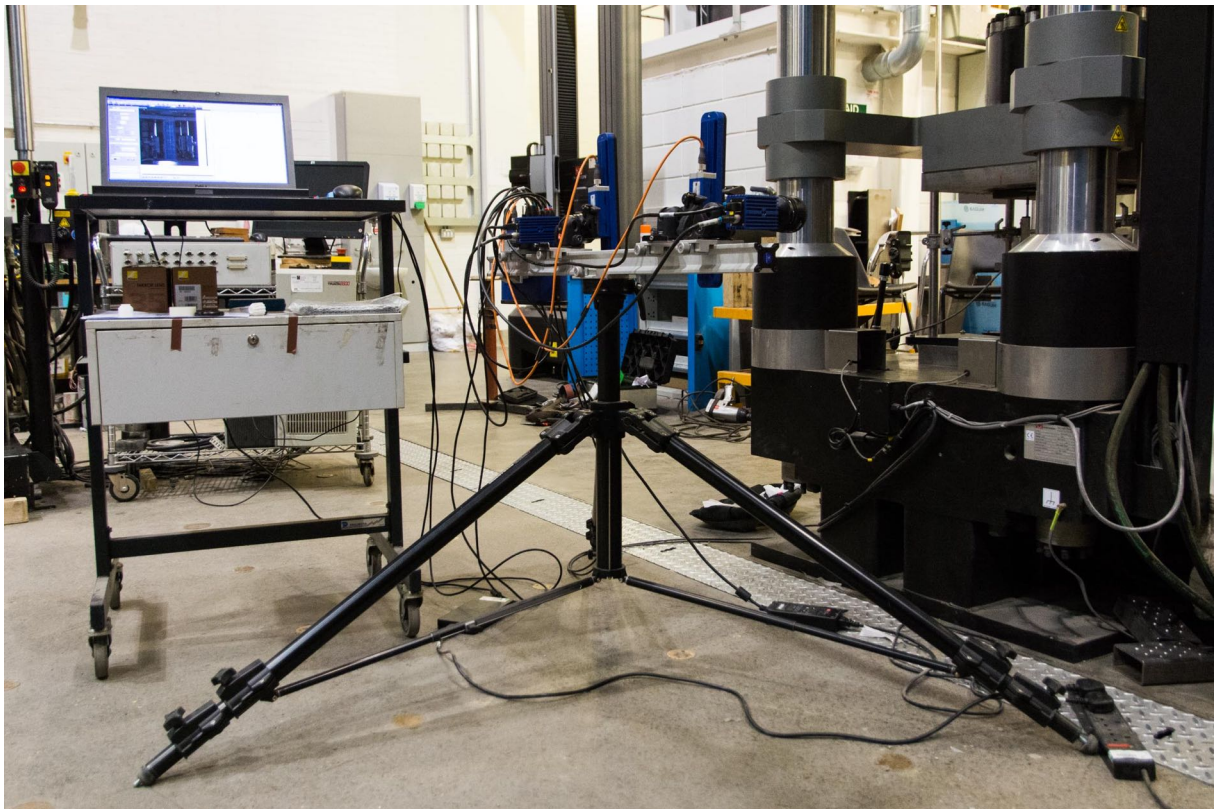


Figure 6.28: The digital image correlation (DIC) hardware setup.

Table 6.8: Summary of stub column tests results and comparisons with existing design methods.

Specimen	N_u (kN)	δ_u (mm)	EN 1993-1-4 (2015) compression class	$c/(t\varepsilon)$	Local slenderness $\bar{\lambda}_p$	$N_u/(A\sigma_{0.2})$	N_u/N_{EC3}	N_u/N_{csm}
50×50×1	42.8	0.81	4	63.9	1.12	0.51	0.87	-
50×50×2	202.3	1.66	1	30.5	0.54	1.29	1.29	1.26
50×50×3	323.4	5.01	1	18.1	0.32	1.44	1.44	1.17
50×50×4	466.2	13.54	1	11.7	0.21	1.63	1.63	1.33
50×50×5	620.7	22.02	1	7.6	0.13	1.86	1.86	1.51
						Average	1.42	1.32
						COV	0.26	0.11

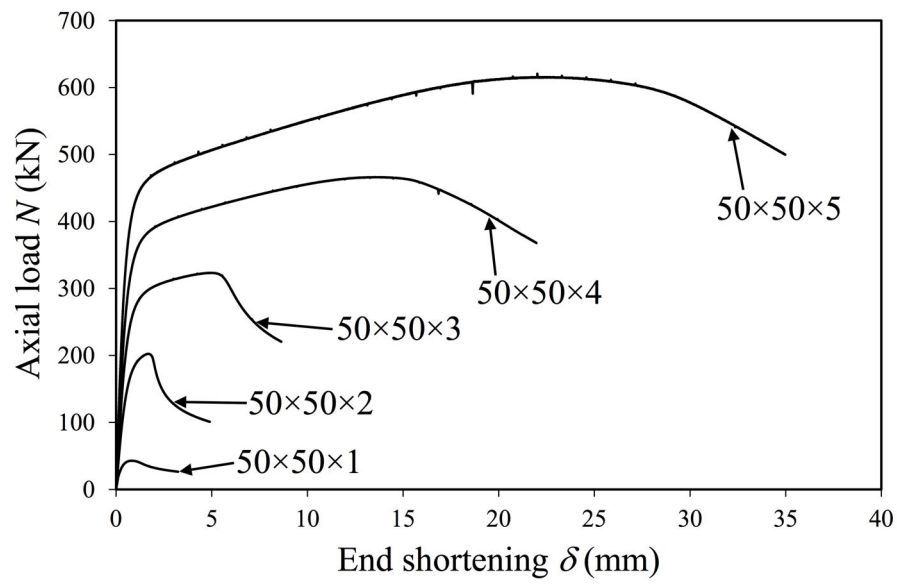


Figure 6.29: Load end-shortening curves for the stub column tests.



Figure 6.30: The deformed 50×50×1, 50×50×2, 50×50×3, 50×50×4 and 50×50×5 SHS stub columns, from left to right.

6.3 Comparison with existing design methods

In this section, the suitability of existing structural design methods for PBF manufactured stainless steel stub SHS columns is appraised. In the comparisons presented, the material properties are taken as the average of the vertical 316L tensile coupons (316L-21 and 316L-22) and all safety factors are taken to be equal to unity.

6.3.1 Comparisons with EN 1993-1-4 (2015) design provisions

The EN 1993-1-4 (2015) guidance for the compression resistance of stainless steel cross-sections is based upon the same approach as for structural carbon steel in EN 1993-1-1 (2014). The following are presented in Table 6.8 — the EN 1993-1-4 (2015) local slenderness parameter $c/(t\varepsilon)$, where c is the element width, t is the thickness and $\varepsilon = [(235/\sigma_{0.2}) (E/210000)]^{0.5}$ is the material parameter, the cross-section class and the ultimate load normalised by the yield load $N_u/(A\sigma_{0.2})$. $N_u/(A\sigma_{0.2})$ is also plotted against the slenderness parameter $c/(t\varepsilon)$ in Figure 6.31 along with a linear regression line. The class 3 limit of $c/(t\varepsilon) = 37$ in EN 1993-1-4 (2015) is the slenderness limit up to which the cross-section is considered to be fully effective in compression, i.e. the cross-section can attain the yield load $A\sigma_{0.2}$. Beyond this limit the cross-section capacity is reduced to allow for the occurrence of failure local buckling prior to reaching the yield load. This limit, and the slightly modified CSM yield slenderness limit of $c/(t\varepsilon) = 38.6$ (Afshan and Gardner, 2013b), may be seen to be generally applicable to PBF manufactured stainless steel SHS.

The PBF stub column results have also been compared with existing, conventionally formed austenitic (Rasmussen and Hancock, 1993a; Talja and Salmi, 1995; Kuwamura, 2003; Liu and Young, 2003; Gardner and Nethercot, 2004a; Young and Lui, 2005; Gardner et al., 2006; Lam and Gardner, 2008; Yuan et al., 2014), duplex (Young and Lui, 2005; Lam and Gardner, 2008; Theofanous and Gardner, 2009; Yuan et al., 2014) and

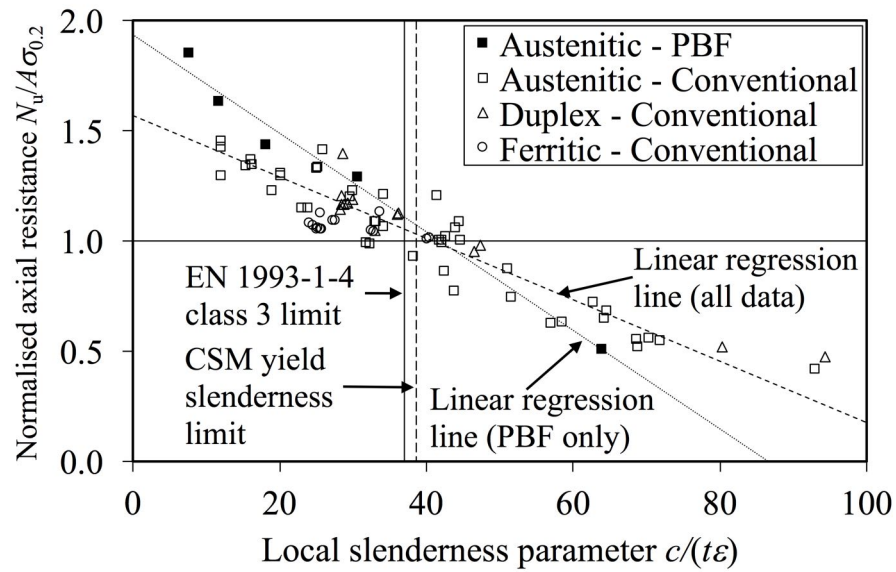


Figure 6.31: Normalised ultimate axial resistance varying with local slenderness.

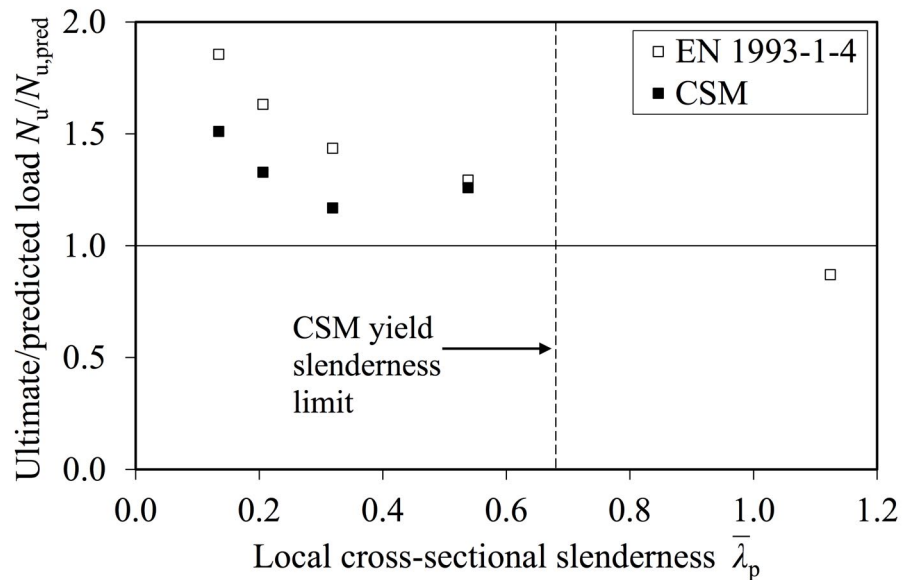


Figure 6.32: EN 1993-1-4 (2015) and CSM compression resistance prediction comparison.

ferritic (Afshan and Gardner, 2013a; Zhao et al., 2015e; Bock et al., 2015a; Arrayago and Real, 2015; Zhao et al., 2016b) SHS stub column data, as shown in Figure 6.31. For the existing SHS dataset, $\sigma_{0.2}$ is taken as the weighted average 0.2% proof stress to account for the increased strength in the cold worked corner regions, where corner material properties are reported. Where corner properties have not been reported, the predictive model of Rossi et al. (2013) has been employed. The non-slender class 1 PBF stub columns all lie towards the upper end of the $N_u/(A\sigma_{0.2})$ range relative to the existing data, while the slender class 4 stub column lies slightly below the existing test data of comparable slenderness, this may be the result of internal defects or residual stresses from the manufacturing method. By fitting a linear regression line to the full stainless steel SHS dataset it can be seen that the current class 3 limit is generally appropriate.

The EN 1993-1-4 (2015) cross-sectional resistance can also be calculated, which for non-slender class 1 cross-sections is the product of the gross cross-sectional area A and the 0.2% proof stress $\sigma_{0.2}$, while for slender class 4 cross-sections the effective area A_{eff} is employed. The ultimate load normalised by the EN 1993-1-4 (2015) prediction N_u/N_{EC3} is reported in Table 6.8 and plotted in Figure 6.32, and it can be seen that the class 1 predictions are very conservative showing that there is additional structural efficiency to be sought, although the class 4 prediction is slightly unconservative. Overall, EN 1993-1-4 (2015) can be observed to be safe, if inefficient, for non-slender PBF SHS design, although further investigation is required for slender cross-sections.

6.3.2 Comparisons with the continuous strength method design provisions

The continuous strength method (CSM) is a deformation based design approach that has been developed to reflect better the observed characteristics of metallic structural elements. Cross-section classification is replaced with a continuous relationship between cross-section slenderness and deformation capacity and the traditional elastic perfectly-

plastic material models are replaced with strain hardening material models. The development of the CSM was outlined in Section 2.3.2. The CSM has recently been set-out for non-slender austenitic, duplex and ferritic stainless steel plated cross-sections (Afshan and Gardner, 2013b; Bock et al., 2015b).

The local slenderness of the cross-section $\bar{\lambda}_p$ can be calculated using Equation 3.1 from Section 3.2, where σ_{cr} is the elastic critical buckling stress of the cross-section, but can be determined conservatively on the basis of the most slender plate in the cross-section (EN 1993-1-5 (2006b)).

The allowable deformation capacity, the strain ratio $\varepsilon_{csm}/\varepsilon_y$, can be determined from the base curve described by

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{0.25}{\bar{\lambda}_p^{3.6}} \text{ for } \bar{\lambda}_p \leq 0.68 \text{ but } \frac{\varepsilon_{csm}}{\varepsilon_y} \leq \min \left(15, \frac{C_1 \varepsilon_u}{\varepsilon_y} \right) \quad (6.1)$$

with $C_1 = 0.1$, which applies to non-slender cross-sections with a local slenderness below the yield slenderness limit of $\bar{\lambda}_p \leq 0.68$ for plated cross-sections. The existing yield slenderness limit can be seen to be suitable from the PBF linear regression line in Figure 6.31.

The CSM limiting material stress f_{csm} can be calculated from Equation 3.26 in Section 3.8, where E_{sh} is the strain hardening modulus and ε_y is the yield strain ($\sigma_{0.2}/E$). The austenitic stainless steel strain hardening modulus E_{sh} and material strain ε_u corresponding to the ultimate tensile stress σ_u can be determined from Equations 3.31 and 3.32 respectively from Section 3.8.3, with $C_2 = 0.16$. These equations can also be applied to duplex stainless steel cross-sections, but modified coefficients should be used for ferritic stainless steels (Bock et al., 2015b).

The CSM cross-sectional axial resistance N_{csm} can then be determined from

$$N_{\text{csm}} = Af_{\text{csm}} \text{ for } \bar{\lambda}_p \leq 0.68. \quad (6.2)$$

The CSM compressive capacity cannot be calculated for the $50 \times 50 \times 1$ SHS stub column as it is a slender cross-section and has a local slenderness greater than the yield slenderness limit. The ultimate resistances normalised by the CSM resistances N_u/N_{csm} and average values of the normalised CSM and EN 1993-1-4 (2015) resistance predictions along with coefficient of variation (COV) values are reported in Table 6.8. The two are also plotted together in Figure 6.32. It can be observed that the CSM is more accurate and consistent in its resistance predictions than existing methods, this is expected as the CSM allows for material strain hardening. Overall the CSM can be seen to offer resistance predictions on the safe side for non-slender PBF SHS, although further work is required to extend it to cover slender cross-sections.

6.4 Conclusions

An experimental programme has been undertaken to investigate the behaviour of additive manufactured metallic material and cross-sections, and to assess the applicability of existing metallic design methods. Stainless steel PH1 and 316L tensile coupons and 316L compression coupons, built using powder bed fusion (PBF), have been tested in a range of building orientations. For the 316L stainless steel coupons it was observed that the 0.2% proof stress $\sigma_{0.2}$ (in both tension and compression) and ultimate tensile stress σ_u decrease in magnitude with increasing building orientation angle; this is attributed to elongated grains in the vertical direction resulting in a lower strength when loaded in this orientation. The magnitude of the measured strengths were also found to be generally higher than conventionally formed material; this is owing to the rapid cooling that occurs in the thin layers of built up material. For the PH1 stainless steel coupons, the 0.2% proof stress $\sigma_{0.2}$ was seen to be fairly uniform, while the ultimate tensile stress σ_u decreased with increasing building angle. The measured Young's moduli E were found

to have no discernible variation with building angle for both the PH1 and 316L stainless steel, but were lower than for the conventionally produced material. The fracture strain showed variation (more than conventional material), with no clear trend for the building angle for the 316L stainless steel, but reduced with increasing building angle for the PH1 material.

A range of additive manufactured 316L stainless steel square hollow section (SHS) stub columns with varying wall thicknesses were also tested. These are believed to be the first structural experiments on additive manufactured metallic cross-sections. The structural behaviour was broadly similar to conventionally produced stainless steel SHS, with the more slender sections showing increased propensity to local buckling. The results were also compared with existing design methods, which were found to generally be applicable to PBF manufactured sections. PBF additive manufacturing is a promising new manufacturing technique that while currently rather slow and expensive, with anticipated developments in manufacturing technology and further advances in understanding of the response of the resulting structural elements, could become an important part of structural engineering in the future.

Chapter 7

Conclusions and suggestions for future work

The testing, simulation and design of conventionally produced and additively manufactured stainless steel hollow structural cross-sections have been the focus of this thesis. This final chapter summarises the major findings and conclusions of this study, and provides recommendations for further work to build upon the findings and conclusions.

7.1 Conclusions

The literature review chapter provided an overview of the key research work relevant to stainless steel circular hollow sections (CHS) and metallic additive manufacturing. Stainless steel was seen to possess a number of unique properties and requirements compared with carbon steel, namely its corrosion resistance, rounded stress–strain material response and more stringent fabrication demands to ensure a high quality surface finish, with aesthetics typically an important factor in the specification of stainless steel. The development of stainless steel design standards was presented, which is of particular importance as efficient design rules are required to mitigate against the high initial cost of stainless steel, the main barrier preventing its wider adoption in construction. The primary focus of this thesis was on the evaluation of existing design

rules and the development of safe, efficient and reliable design guidance for stainless steel CHS. The secondary objective was the investigation of the structural performance of metallic additively manufactured elements and the appraisal of current design approaches for sections produced using this novel manufacturing technique.

Chapter 3 introduced the experimental programme, which encompassed tensile coupon tests, 10 stub column tests, 37 flexural buckling tests, 12 combined loading cross-section tests and 10 beam-column member tests on austenitic, duplex and ferritic stainless steel CHS, and the results are used throughout Chapters 3 to 5. The cross-section behaviour of CHS was then investigated using a structural carbon steel, stainless steel and aluminium CHS dataset collected from the literature for pure compression and bending, with the new stainless steel stub column results supplementing the existing dataset. It was seen that current design methods are overly conservative in their predictions of cross-section resistance through the adoption of cross-section classification and by limiting the maximum material strength to the yield stress. The continuous strength method (CSM) was extended to cover metallic CHS under pure compression and bending, through the identification of an appropriate yield slenderness limit, below which significant benefit can be sought from strain hardening and delineating non-slender and slender CHS, the calibration of the non-slender and slender base curves, linking the local slenderness to the allowable deformation capacity, the application of existing material models and the development of suitable resistance functions. The CSM was found to offer generally more accurate, between about 5% and 20%, and consistent predictions of cross-section resistance compared to existing design methods.

The flexural buckling resistance of stainless steel CHS was investigated in Chapter 4 following prior observations that the EN 1993-1-4 (2015) flexural buckling curve can be unconservative in its capacity predictions. A dataset of existing long column stainless steel CHS tests was collated and was seen to be very limited, with only a small number of existing austenitic CHS long column tests and no duplex or ferritic CHS long column tests;

this was expanded with new experimental and numerical data on austenitic, duplex and ferritic stainless steel CHS columns, covering cross-section classes 1–4 and a wide range of global slenderness values. Existing non-iterative design proposals were then evaluated and were typically found to offer improved predictions of member level resistance. A new flexural buckling curve was proposed and it was seen to generally provide safe and efficient predictions of the member level compressive resistance when using the current EN 1993-1-4 (2015) partial safety factor.

Combined loading tests at both the cross-section and member level and their subsequent design was the focus of Chapter 5. EN 1993-1-4 (2015) does not currently have guidance for cross-section classification of CHS under combined loading and an expression utilising linear interpolation between the compressive and bending class 3 slenderness limits was proposed. The ultimate loads were compared against the current European design guidance and it was observed that the current design approach is rather conservative at the cross-section level and for members with a high ratio of bending moment to axial load, while it is unconservative for members with a low ratio of bending moment to axial load. A previous extension of the continuous strength method (CSM) to stainless steel CHS cross-sections under combined loading was found to offer improvements over the current European guidance. Numerical models of the test specimens were produced and a parametric study was undertaken for stainless steel beam-columns. More efficient and appropriate interaction factors and end points for stainless steel CHS beam-columns were proposed and statistical analyses carried out to ensure that the beam-column resistance predicted is safe and reliable.

In Chapter 6, an experimental programme was reported that investigated the behaviour of additive manufactured stainless steel material and cross-sections, and assessed the applicability of existing metallic design methods. Stainless steel PH1 and 316L tensile coupons and 316L compression coupons were built in a range of orientations using powder bed fusion (PBF). Anisotropic behaviour was observed for the 0.2% proof stress

$\sigma_{0.2}$ and ultimate tensile stress σ_u , due to elongated grains in the vertical direction resulting in a lower strength. The fracture strain ε_f showed some variation with building orientation and the Young's moduli E were found to have no discernible change with the building angle. The magnitude of the measured strengths were found to be generally higher than for the conventionally formed material, which is attributed to the rapid cooling that occurs in the thin layers of built up material, though the Young's modulus was lower. For the 316L stainless steel square hollow section (SHS) stub columns with varying wall thicknesses that were also built and tested, these are thought to be the first structural experiments on additive manufactured metallic cross-sections. Existing test results on conventionally produced stainless steel SHS stub columns were collated from the literature and were seen to have generally similar structural behaviour to the PBF specimens. The resistance predictions from EN 1993-1-4 (2015) and the CSM were calculated and these design methods were generally found to be applicable to PBF manufactured sections, although for the most slender cross-section EN 1993-1-4 (2015) was unconservative.

The ultimate aim of this study was to ensure that structural design for stainless steel CHS is safe and efficient. At the cross-section level, the pure compression and bending resistance is generally more accurately and consistently predicted with the CSM for structural carbon steel, stainless steel and aluminium CHS. At the member level a new safe and reliable stainless steel CHS flexural buckling curve is proposed to replace the current EN 1993-1-4 (2015) curve which is unconservative for certain global slenderness values; for stainless steel CHS beam-columns more efficient interaction factors are proposed to replace the codified interaction factors that can be either conservative or unconservative depending upon the ratio of axial load to bending moment. Improved predictions of stainless steel CHS resistance will lead to lighter structures with more efficient material use, leading to more sustainable construction, while safe and reliable predictions will help to prevent unacceptable structural failures.

The secondary aim of this thesis was to explore the potential of metallic additive manufacturing in the construction sector and to appraise the suitability of existing design methods to cross-sections produced through this manufacturing technique. Power bed fusion (PBF) metallic additive manufacturing was observed to be a promising new manufacturing method for structural cross-sections, with material strength advantages over conventionally produced stainless steel and existing design approaches were found to be generally applicable. PBF has the potential to become an important part of structural engineering in the future, following an improved understanding of the response of the resulting structural elements and further advances in the manufacturing technology.

7.2 Suggestions for further work

The cross-section behaviour of metallic CHS was investigated in Chapter 3. The existing stainless steel CHS stub column dataset was supplemented with additional experimental results. The length of the CHS stub columns was short enough to prevent global buckling, but sufficiently long to include representative local imperfections and residual stresses; it was noted that there is no CHS specific guidance on length effects other than the general guidance that the member length of any cold-formed structural cross-section should be greater than three times the maximum cross-section dimension and less than 20 times the least radius-of-gyration (Ziemian, 2010). Stub column tests could be undertaken with a range of lengths to determine more definitive recommendations for the length of CHS stub columns. The stainless steel and aluminium CHS four-point bending datasets are relatively small compared to the other metallic materials and there is scope for additional bending tests to be undertaken. The CSM material models are another avenue for further research — in particular a hot-finished structural carbon steel cross-section that takes advantage of the strain hardening material properties beyond the yield plateau is required, and there may also be scope with the other metallic material models for additional improvements over those currently attained. The CHS extension of the CSM

could be applied to elliptical hollow sections (EHS) due to the similarity in cross-sections, with a continuously curved perimeter, which, while not optimised for this cross-section, may offer improved predictions and reduced scatter compared with existing design methods.

A new, safe, efficient and reliable stainless steel CHS flexural buckling curve was proposed in Chapter 4 based on existing and new experimental and numerical column data. The new experimental tests were limited by the maximum extension of the testing rig, which for the austenitic and duplex columns at ICL allowed a maximum global slenderness of $\bar{\lambda} = 1.65$ with the chosen cross-sections and material grades. It would be useful to have experimental data for more globally slender stainless steel CHS columns (the most slender CHS column from the literature had a global slenderness of $\bar{\lambda} = 1.81$), and more finite element results could be produced. In Chapter 4, there were a number of fixed end column tests from Young and Hartono (2002) which had a lower ultimate resistance than would be expected; it would be prudent to attempt similar fixed ended column tests to investigate further their behaviour.

Chapter 5 explored the structural behaviour of stainless steel CHS under combined loading at both the local and member level. In the literature, there has been no experimental study on duplex stainless steel CHS members under combined axial load and bending moment; thus there is scope for additional work to close this gap.

The implications of metallic additive manufacturing in the construction sector was tentatively explored in Chapter 6, and it is readily apparent that there is significant potential for additional research in this fairly new field. The tensile and compressive coupon tests comprised of only one coupon per considered orientation and it is clear that repeat coupons should be tested to allow more definitive conclusions to be made. Additional tensile and compressive coupons and stub columns could be built using equipment and powder from different manufacturers, or even alternative metallic

additive manufacturing methods such as directed energy deposition (DED), to provide useful insight into the repeatability of the physical properties, which is of importance when determining the applicability of design rules for a given manufacturing technique. The rapid cooling that is inherent to powder bed fusion can result in significant internal residual stresses and it would be beneficial to measure these, particularly for very thin-walled cross-sections, and it would also be useful to X-ray image future thin-walled cross-sections to attempt to quantify the extent of internal defects. From the literature it is unclear if the anisotropy observed in the 316L stainless steel tensile and compressive coupons can be reduced through heat treatment, and this is another potential area for additional research, particularly if the higher strength, compared with conventionally produced stainless steel, can be maintained. Square hollow sections (SHS) stub columns were tested and the results compared with those obtained from conventionally produced stainless steel SHS; for completeness it would be advisable to build, test and compare additively manufactured PBF CHS, rectangular hollow sections (RHS), EHS, I sections, angles and channels. The 350 mm cube build envelope of PBF could be expanded by welding smaller member lengths together, which could also be the focus of future research, especially comparing additive manufactured structural members to conventionally formed metallic members. Building techniques such as DED could also be used to manufacture more typical full-sized structural members. The SHS stub columns tested in this study, while utilising a novel manufacturing technique, do not take advantage of the geometric freedom that additive manufacturing allows. A clear path for future metallic research in this field would be to build and test structural cross-sections and members which could only be manufactured using these techniques and that offer significant strength, ductility or weight savings over traditional structural elements. This could take the form of modifying the porosity to reduce weight where material is not required, varying the wall thickness along the member length in hollow sections or having internal structures within hollow sections that are difficult or cost prohibitive to fabricate conventionally.

References

- Abe, F., Osakada, K., Shiomi, M., Uematsu, K., and Matsumoto, M. (2001). The manufacturing of hard tools from metallic powders by selective laser melting. *Journal of Materials Processing Technology*, 111(1-3):210–213.
- Afshan, S., Francis, P., Baddoo, N. R., and Gardner, L. (2015). Reliability analysis of structural stainless steel design provisions. *Journal of Constructional Steel Research*, 114:293–304.
- Afshan, S. and Gardner, L. (2013a). Experimental study of cold-formed ferritic stainless steel hollow sections. *Journal of Structural Engineering ASCE*, 139:717–728.
- Afshan, S. and Gardner, L. (2013b). The continuous strength method for structural stainless steel design. *Thin-Walled Structures*, 68:42–49.
- Afshan, S., Rossi, B., and Gardner, L. (2013). Strength enhancements in cold-formed structural sections - part I: material testing. *Journal of Constructional Steel Research*, 83:177–188.
- Agarwala, M., Bourell, D., Beaman, J., Marcus, H., and Barlow, J. (1995). Post-processing of selective laser sintered metal parts. *Rapid Prototyping Journal*, 1(2):36–44.
- American Institute of Steel Construction (2010). *ANSI/AISC 360-10 specification for structural steel buildings*.
- American Institute of Steel Construction (2013). *Design guide 27: structural stainless steel*.

- American Iron and Steel Institute (1968). *Specification for the design of light gauge cold-formed stainless steel structural members*.
- American Iron and Steel Institute (1974). *Stainless steel cold-formed structural design manual*. Washington, D.C.
- American Iron and Steel Institute (2004). *Commentary on appendix 1 design of cold-formed steel structural members with the direct strength method*.
- American Society of Civil Engineers (2002). *Specification for the design of cold-formed stainless steel structural members [SEI/ASCE 8-02], [Standards No. 02-008]*. New York.
- Anzalone, G. C., Wijnen, B., Sanders, P. G., and Pearce, J. M. (2013). A low-cost open-source metal 3-D printer. *IEEE Access*, 1:803–810.
- Arrayago, I. and Real, E. (2015). Experimental study on ferritic stainless steel RHS and SHS cross-sectional resistance under combined loading. *Structures*, 4:69–79.
- Arrayago, I. and Real, E. (2016). Experimental study on ferritic stainless steel simply supported and continuous beams. *Journal of Constructional Steel Research*, 119:50–62.
- Arrayago, I., Real, E., and Gardner, L. (2015). Description of stress-strain curves for stainless steel alloys. *Materials & Design*, 87:540–552.
- Arrayago, I., Real, E., and Mirambell, E. (2016). Experimental study on ferritic stainless steel RHS and SHS beam-columns. *Thin-Walled Structures*, 100:93–104.
- Arup (2015). 3D makeover for hyper-efficient metalwork. http://www.arup.com/news/2015_05_may/11_may_3d_makeover_for_hyper-efficient_metalwork [Accessed 10th October 2016].
- Ashraf, M., Gardner, L., and Nethercot, D. A. (2005). Strength enhancement of the corner regions of stainless steel cross-sections. *Journal of Constructional Steel Research*, 61:37–52.

- Ashraf, M., Gardner, L., and Nethercot, D. A. (2006a). Compression strength of stainless steel cross-sections. *Journal of Constructional Steel Research*, 62:105–115.
- Ashraf, M., Gardner, L., and Nethercot, D. A. (2006b). Finite element modelling of structural stainless steel cross-sections. *Thin-Walled Structures*, 44(10):1048–1062.
- Ashraf, M., Gardner, L., and Nethercot, D. a. (2008a). Resistance of stainless steel CHS columns based on cross-section deformation capacity. *Journal of Constructional Steel Research*, 64(9):962–970.
- Ashraf, M., Gardner, L., and Nethercot, D. a. (2008b). Structural stainless steel design: resistance based on deformation capacity. *Journal of Structural Engineering ASCE*, 134(3):402–411.
- ASTM International (2013). *Standard Terminology for Additive Manufacturing Technologies*.
- ASTM International (2015). *Standard terminology for additive manufacturing technologies - general principles - terminology*.
- ASTM International (2016). *E8/E8M standard test methods for tension testing of metallic materials*.
- Autodesk (2015a). AutoCAD 2015 for Mac.
- Autodesk (2015b). Netfabb.
- Ayrton, W. E. and Perry, J. (1886). On struts. *The Engineer*, 62:464–465 and 513–514.
- Baddoo, N. (2013). 100 years of stainless steel: a review of structural applications and the development of design rules. *The Structural Engineer*, 91(8):10–18.
- Baddoo, N. R. (2008). Stainless steel in construction: a review of research, applications, challenges and opportunities. *Journal of Constructional Steel Research*, 64(11):1199–1206.

- Baddoo, N. R. and Burgan, B. A. (2001). Structural design of stainless steel. Technical report, The Steel Construction Institute.
- Bardi, F. C. and Kyriakides, S. (2006). Plastic buckling of circular tubes under axial compression - part I: experiments. *International Journal of Mechanical Sciences*, 48(8):830–841.
- Baufeld, B., Van der Biest, O., and Gault, R. (2010). Additive manufacturing of Ti-6Al-4V components by shaped metal deposition: microstructure and mechanical properties. *Materials and Design*, 31:S106–S111.
- BBC News (2016). Tomorrow's buildings: construction industry goes robotic. <http://www.bbc.co.uk/news/technology-35746648> [Accessed 10th October 2016].
- Becque, J., Lecce, M., and Rasmussen, K. J. R. (2008). The direct strength method for stainless steel compression members. *Journal of Constructional Steel Research*, 64(11):1231–1238.
- Blachut, J. and Galletly, G. D. (1995). Buckling strength of imperfect steel hemispheres. *Thin-Walled Structures*, 23(1-4):1–20.
- Blachut, J., Galletly, G. D., and Moffat, D. G. (1991). An experimental and numerical study into the collapse strength of steel domes. *Buckling of Shell Structures, on Land, in the Sea and in the Air*, pages 344–358.
- Bock, M., Arrayago, I., and Real, E. (2015a). Experiments on cold-formed ferritic stainless steel slender sections. *Journal of Constructional Steel Research*, 109:13–23.
- Bock, M., Gardner, L., and Real, E. (2015b). Material and local buckling response of ferritic stainless steel sections. *Thin-Walled Structures*, 89:131–141.
- Bogue, R. (2013). 3D printing: the dawn of a new era in manufacturing? *Assembly Automation*, 33(4):307–311.

- Boissonnade, N., Greiner, R., Jaspart, J. P., and Lindner, J. (2006). Rules for member stability in EN 1993-1-1: background documentation and design guidelines. Technical report, ECCS European Convention for Constructional Steelwork, New York.
- British Standards Institution (2000). *BS 5950-1:2000 structural use of steelwork in building - part 1: code of practice for design - rolled and welded sections*.
- British Standards Institution (2014a). *BS EN 10088-1:2014 stainless steels part 1: list of stainless steels*.
- British Standards Institution (2014b). *BS EN 10088-2:2014 stainless steels part 2: technical delivery conditions for sheet/plate and strip of corrosion resisting steels for general purposes*.
- BSSA. Stainless steel in swimming pool buildings.
- Buchanan, C., Gardner, L., and Liew, A. (2015). The continuous strength method for circular hollow sections. In Batista, E., Vellasco, P., and Lima, L., editors, *Proceedings of the 15th International Symposium on Tubular Structures*, pages 621–628, Rio de Janeiro. Balkema.
- Buchanan, C., Gardner, L., and Liew, A. (2016a). The continuous strength method for the design of circular hollow sections. *Journal of Constructional Steel Research*, 118:207–216.
- Buchanan, C., Matilainen, V.-P., Salminen, A., and Gardner, L. (2016b). Testing of additive manufactured metallic coupons and cross-sections. In Young, B., editor, *Proceedings of the Eight International Conference on Steel and Aluminium Structures*, Hong Kong.
- Buchanan, C., Matilainen, V.-P., Salminen, A., and Gardner, L. (2017). Structural performance of additive manufactured metallic material and cross-sections. *Journal of Constructional Steel Research*.

- Buchanan, C., Real, E., and Gardner, L. (2016c). Beam-column behaviour of ferritic stainless steel CHS members. In Schafer, B., editor, *Proceedings of the 7th International Conference on Coupled Instabilities in Metal Structures*.
- Buchanan, C., Real, E., and Gardner, L. (2016d). Testing and design of stainless steel CHS columns. In Young, B., editor, *Proceedings of the Eight International Conference on Steel and Aluminium Structures*, Hong Kong.
- Burgan, B. A., Baddoo, N. R., and Gilsenan, K. A. (2000). Structural design of stainless steel members - comparison between Eurocode 3, Part 1.4 and test results. *Journal of Constructional Steel Research*, 54(1):51–73.
- Centre for Advanced Structural Engineering (1990). *Compression tests of stainless steel tubular columns. Investigation report S770*.
- Cesaretti, G., Dini, E., De Kestelier, X., Colla, V., and Pambaguian, L. (2014). Building components for an outpost on the lunar soil by means of a novel 3D printing technology. *Acta Astronautica*, 93:430–450.
- Chan, T. M. and Gardner, L. (2008a). Bending strength of hot-rolled elliptical hollow sections. *Journal of Constructional Steel Research*, 64(9):971–986.
- Chan, T. M. and Gardner, L. (2008b). Compressive resistance of hot-rolled elliptical hollow sections. *Engineering Structures*, 30(2):522–532.
- Chen, W. F. and Ross, D. A. (1977). Tests of fabricated tubular columns. *Journal of the Structural Division*, 100(3):619–634.
- China Association for Engineering Construction Standardisation (2015). *CECS 410:2015 Technical specification for stainless steel structures*.
- Coetzee, J., Van den Berg, G., and Van der Merwe, P. (1990). The effect of work hardening and residual stresses due to cold-work of forming on the strength of cold-formed stainless steel lipped channel section. In *Tenth international speciality conference on cold-formed steel structures*, pages 143–162, St. Louis, Missouri, USA. AISI.

- Cooper, D. E., Stanford, M., Kibble, K. A., and Gibbons, G. J. (2012). Additive manufacturing for product improvement at Red Bull Technology. *Materials and Design*, 41:226–230.
- Cruise, R. B. and Gardner, L. (2008a). Residual stress analysis of structural stainless steel sections. *Journal of Constructional Steel Research*, 64(3):352–366.
- Cruise, R. B. and Gardner, L. (2008b). Strength enhancements induced during cold forming of stainless steel sections. *Journal of Constructional Steel Research*, 64:1310–1316.
- Dassault Systèmes (2015a). *Abaqus analysis user's guide, Abaqus 2016*.
- Dassault Systèmes (2015b). Solidworks.
- Dassault Systèmes Simulia Corp. (2013). *Abaqus 6.13 getting started with Abaqus: interactive edition*.
- Dassault Systèmes Simulia Corp. (2015). *Abaqus/CAE 2016*.
- Donnell, L. H. (1934). A new theory for the buckling of thin cylinders under axial compression and bending. *Transactions of the American Society of Mechanical Engineers*, 56:795–806.
- Dutta, D. (2002). *Structures with hollow sections*. Wiley VCH, Weinheim.
- El-Zanaty, M. H. and Murray, D. W. (1983). Nonlinear finite element analysis of steel frames. *Journal of Structural Engineering ASCE*, 109(2):353–368.
- Elchalakani, M., Zhao, X. L., and Grzebieta, R. (2002a). Bending tests to determine slenderness limits for cold-formed circular hollow sections. *Journal of Constructional Steel Research*, 58(11):1407–1430.
- Elchalakani, M., Zhao, X. L., and Grzebieta, R. (2002b). Tests on concrete filled double-skin (CHS outer and SHS inner) composite short columns under axial compression. *Thin-Walled Structures*, 40(5):415–441.

- Ellobody, E. (2013). Numerical modelling of fibre reinforced concrete-filled stainless steel tubular columns. *Thin-Walled Structures*, 63:1–12.
- Ellobody, E. and Young, B. (2005). Structural performance of cold-formed high strength stainless steel columns. *Journal of Constructional Steel Research*, 61(12):1631–1649.
- Ellobody, E. and Young, B. (2007). Investigation of cold-formed stainless steel non-slender circular hollow section columns. *Steel and Composite Structures*, 7(4):321–337.
- EOS (2014). *Material data sheet EOS stainless Steel 316L*.
- EOS (2015). *Material data sheet EOS stainless steel PH1 for EOS M 290*.
- Euro Inox (1993). *Design manual for structural stainless steel (first edition)*.
- European Committee for Standardisation (1996). *ENV 1993-1-4 Eurocode 3 - design of steel structures - part 1.4: general rules - supplementary rules for stainless steel*.
- European Committee for Standardisation (2005). *BS EN 1990:2002+A1:2005 Eurocode - basis of structural design*.
- European Committee for Standardisation (2006a). *BS EN 10219-2:2006 Cold formed welded structural hollow sections of non-alloy and fine grain steels. Part 2: tolerances, dimensions and sectional properties*, volume 3.
- European Committee for Standardisation (2006b). *EN 1993-1-5:2006 Eurocode 3: design of steel structures - part 1-5: plated structural elements*. CEN, Brussels.
- European Committee for Standardisation (2007). *EN 1999-1-1:2007 Eurocode 9: design of aluminium structures - part 1-1: general structural rules*.
- European Committee for Standardisation (2009). *BS EN ISO 6892-1:2009 metallic materials - tensile testing part 1: method of test at ambient temperature*.
- European Committee for Standardisation (2010). *EN 1993-1-3:2006 Eurocode 3 - design of steel structures - part 1-3: general rules - supplementary rules for cold-formed members and sheeting*.

- European Committee for Standardisation (2014). *EN 1993-1-1:2005+A1:2014 Eurocode 3: design of steel structures - part 1-1: general rules and rules for buildings.*
- European Committee for Standardisation (2015). *EN 1993-1-4:2006+A1:2015 Eurocode 3 - design of steel structures - part 1-4: general rules - supplementary rules for stainless steel.*
- European Committee for Standardisation (CEN) (2006). *EN 1993-1-4:2006 Eurocode 3 - design of steel structures - part 1-4: general rules - supplementary rules for stainless steel.*
- European Committee for Standardisation (CEN) (2010). *EN 1993-1-8:2005 Eurocode 3: design of steel structures - part 1-8: design of joints.*
- Facchini, L., Magalini, E., Robotti, P., Molinari, A., Höges, S., and Wissenbach, K. (2010). Ductility of a Ti-6Al-4V alloy produced by selective laser melting of prealloyed powders. *Rapid Prototyping Journal*, 16(6):450–459.
- Flügge, W. (1932). Die stabilität der kreiszylinderschale. *Ingenieur-Archiv*, 3:463–506.
- Foster, A. S. J. (2014). *Stability and design of steel beams in the strain-hardening range.* PhD thesis, Imperial College London.
- Foster, A. S. J. and Gardner, L. (2013). Ultimate behaviour of steel beams with discrete lateral restraints. *Thin-Walled Structures*, 72:88–101.
- Foster, A. S. J. and Gardner, L. (2016). Stability of steel beams using the continuous strength method. *Thin-Walled Structures*, 100:1–13.
- Foster, A. S. J., Gardner, L., and Wang, Y. (2015). Practical strain-hardening material properties for use in deformation-based structural steel design. *Thin Walled Structures*, 92:115–129.
- Fratamico, D. C. and Schafer, B. W. (2014). Numerical studies on the composite action and buckling behavior of built-up cold-formed steel columns. In *22nd International Specialty Conference on Recent Research and Developments in Cold-Formed Steel Design and Construction*, pages 213–228.

- Frearson, A. (2016). DUS Architects builds 3D-printed micro home in Amsterdam. <https://www.dezeen.com/2016/08/30/dus-architects-3d-printed-micro-home-amsterdam-cabin-bathtub/> [Accessed 10th October 2016].
- Galjaard, S., Hofman, S., and Ren, S. (2014). New opportunities to optimize structural designs in metal by using additive manufacturing. In *Advances in Architectural Geometry 2014*, pages 79–93. Springer International Publishing.
- Gardner, L. (2002). *A new approach to structural stainless steel design*. PhD thesis, Imperial College of Science, Technology and Medicine.
- Gardner, L. (2005). The use of stainless steel in structures. *Progress in Structural Engineering and Materials*, 7(2):45–55.
- Gardner, L. (2008). The continuous strength method. *Proceedings of the Institution of Civil Engineers: Structures and Buildings*, 161(3):127–133.
- Gardner, L. and Ashraf, M. (2006). Structural design for non-linear metallic materials. *Engineering Structures*, 28(6):926–934.
- Gardner, L. and Baddoo, N. R. (2006). Fire testing and design of stainless steel structures. *Journal of Constructional Steel Research*, 62:532–543.
- Gardner, L., Chan, T. M., and Abela, J. M. (2011a). Structural behaviour of elliptical hollow sections under combined compression and uniaxial bending. *Advanced Steel Construction*, 7(1):86–112.
- Gardner, L., Law, K. H., and Buchanan, C. (2014). Unified slenderness limits for structural steel circular hollow sections. *Romanian Journal of Technical Sciences, Applied Mechanics*, 59(1-2):153–163.
- Gardner, L. and Nethercot, D. A. (2004a). Experiments on stainless steel hollow sections - part 1: material and cross-sectional behaviour. *Journal of Constructional Steel Research*, 60(9):1291–1318.

- Gardner, L. and Nethercot, D. A. (2004b). Experiments on stainless steel hollow sections - part 2: member behaviour of columns and beams. *Journal of Constructional Steel Research*, 60(9):1319–1332.
- Gardner, L. and Nethercot, D. A. (2004c). Numerical modeling of stainless steel structural components - a consistent approach. *Journal of Structural Engineering ASCE*, 130(10):1586–1601.
- Gardner, L. and Nethercot, D. A. (2004d). Structural stainless steel design: a new approach. *Structural Engineer*, 82(21):2–9.
- Gardner, L., Talja, A., and Baddoo, N. R. (2006). Structural design of high-strength austenitic stainless steel. *Thin-Walled Structures*, 44:517–528.
- Gardner, L., Wang, F., and Liew, A. (2011b). Influence of strain hardening on the behavior and design of steel structures. *International Journal of Structural Stability and Dynamics*, 11(5):855–875.
- Gardner, L., Yun, X., Macorini, L., and Kucukler, M. (2015). The continuous strength method for hot-rolled steel and steel-concrete composite design. In *Proceedings of the Eleventh International Conference on Advances in Steel and Concrete Composite Structures (ASCCS)*, Tsinghua University, Beijing, China.
- Gardner, L., Yun, X., Macorini, L., and Kucukler, M. (2016). Hot-rolled steel and steel-concrete composite design incorporating strain hardening. *Structures*.
- Gerard, G. and Becker, H. (1957). *Handbook of structural stability*. NACA, Washington, D.C.
- Giakoumelis, G. and Lam, D. (2004). Axial capacity of circular concrete-filled tube columns. *Journal of Constructional Steel Research*, 60(7):1049–1068.
- Girão Coelho, A. M., Simão, P. D., and Wadee, M. A. (2013). Imperfection sensitivity of column instability revisited. *Journal of Constructional Steel Research*, 90:265–282.

- Greiner, R. and Kettler, M. (2008). Interaction of bending and axial compression of stainless steel members. *Journal of Constructional Steel Research*, 64(11):1217–1224.
- Greiner, R. and Lindner, J. (2006). Interaction formulae for members subjected to bending and axial compression in Eurocode 3 - the method 2 approach. *Journal of Constructional Steel Research*, 62(8):757–770.
- Gresnigt, A. M. and van Foeken, R. J. (2001). Local buckling of UOE and seamless steel pipes. In Chung, J., Matsui, T., and Moshagen, H., editors, *Proceedings of the Eleventh (2001) International Offshore and Polar Engineering Conference*, pages 131–142, Stavanger.
- Gu, D. D., Meiners, W., Wissenbach, K., and Poprawe, R. (2012). Laser additive manufacturing of metallic components: materials, processes and mechanisms. *International Materials Reviews*, 57(3):133–164.
- Guan, K., Wang, Z., Gao, M., Li, X., and Zeng, X. (2013). Effects of processing parameters on tensile properties of selective laser melted 304 stainless steel. *Materials and Design*, 50:581–586.
- Guo, L., Yang, S., and Jiao, H. (2013). Behavior of thin-walled circular hollow section tubes subjected to bending. *Thin-Walled Structures*, 73:281–289.
- Haedir, J., Bambach, M. R., Zhao, X. L., and Grzebieta, R. H. (2009). Strength of circular hollow sections (CHS) tubular beams externally reinforced by carbon FRP sheets in pure bending. *Thin-Walled Structures*, 47(10):1136–1147.
- Hassanein, M. F. and Silvestre, N. (2013). Flexural behavior of lean duplex stainless steel girders with slender unstiffened webs. *Journal of Constructional Steel Research*, 85:12–23.
- Henke, K. and Treml, S. (2013). Wood based bulk material in 3D printing processes for applications in construction. *European Journal of Wood and Wood Products*, 71(1):139–141.

- Heselmans, J. and Vermeij, P. (2013). Fatal accident in Dutch swimming pool caused by environmentally cracked bolts. In *NACE International Corrosion 2013*, pages 1–13.
- Hill, H. N. (1944). *Determination of stress-strain relations from "offset" yield strength values*. Washington, DC, United States.
- Holshouser, C., Newell, C., Palas, S., Martin, L., Duty, C., Love, L., Kunc, V., Lind, R., Lloyd, P., Rowe, J., Dehoff, R., Peter, W., and Blue, C. (2013). Out of bounds additive manufacturing. *Advanced Materials and Processes*, 171(3):15–17.
- Hradil, P., Fülöp, L., and Talja, A. (2012). Global stability of thin-walled ferritic stainless steel members. *Thin-Walled Structures*, 61:106–114.
- Hradil, P. and Talja, A. (2014). Numerical verification of stainless steel overall buckling curves. *Thin-Walled Structures*, 83:52–58.
- Huang, Y. and Young, B. (2012). Material properties of cold-formed lean duplex stainless steel sections. *Thin-Walled Structures*, 54:72–81.
- Huang, Y. and Young, B. (2013a). Experimental and numerical investigation of cold-formed lean duplex stainless steel flexural members. *Thin Walled Structures*, 73:216–228.
- Huang, Y. and Young, B. (2013b). Tests of pin-ended cold-formed lean duplex stainless steel columns. *Journal of Constructional Steel Research*, 82:203–215.
- Huang, Y. and Young, B. (2014). Experimental investigation of cold-formed lean duplex stainless steel beam-columns. *Thin-Walled Structures*, 76:105–117.
- Huang, Y. and Young, B. (2015). Design of cold-formed lean duplex stainless Steel members in combined compression and bending. *Journal of Structural Engineering ASCE*, 141(5):1–10.
- International Stainless Steel Forum (2007). *The ferritic solution*.
- International Stainless Steel Forum (2010). *Stainless steel in sewage treatment plants*.

- Jandera, M., Gardner, L., and Machacek, J. (2008). Residual stresses in cold-rolled stainless steel hollow sections. *Journal of Constructional Steel Research*, 64(11):1255–1263.
- Jiao, H. and Zhao, X. L. (2003). Imperfection, residual stress and yield slenderness limit of very high strength (VHS) circular steel tubes. *Journal of Constructional Steel Research*, 59(2):233–249.
- Jiao, H. and Zhao, X. L. (2004). Section slenderness limits of very high strength circular steel tubes in bending. *Thin-Walled Structures*, 42(9):1257–1271.
- Jirsa, J. O., Lee, F. H., Wilhoit Jr., J. C., and Merwin, J. E. (1972). Ovaling of pipelines under pure bending. In *Proceedings of the Offshore Technology Conference*, pages 573–579, Dallas.
- Johansson, M. and Gylltoft, K. (2002). Mechanical behavior of circular steel-concrete composite stub columns. *Journal of Structural Engineering ASCE*, 128(8):1073–1081.
- Johnson, A. L. and Winter, G. (1966). Behaviour of stainless steel columns and beams. *Journal of the Structural Division ASCE*, ST5:97–118.
- Joosten, S. K. (2015). Printing a stainless steel bridge: an exploration of structural properties of stainless steel additive manufactures for civil engineering purposes. Technical report, Delft University of Technology.
- Kabir, M. H. and Fawzia, S. (2014). Durability study of CFRP strengthened steel circular hollow sections members under marine environment. In *23rd Australasian Conference on the Mechanics of Structures and Materials*, number December, pages 1–6, Byron Bay, Australia.
- Kahlen, F.-J. and Kar, A. (1999). Tensile strengths for laser-fabricated parts and similarity parameters for rapid manufacturing. *Journal of Manufacturing Science and Engineering*, 123(1):38–44.

- Kamba, T. (1996). Stub column test of high-strength CHS steel column with small diameter-to-thickness ratio. In Farkas, J. and Jarmai, K., editors, *Proceedings of the Seventh International Symposium on Tubular Structures*, pages 397–404, Rotterdam. A.A. Balkema.
- Karren, K. W. (1967). Corner properties of cold-formed shapes. *Journal of the Structural Division ASCE*, 93(1):401–433.
- Kempen, K., Thijs, L., Van Humbeeck, J., and Kruth, J. P. (2012). Mechanical properties of AlSi10Mg produced by selective laser melting. *Physics Procedia*, 39:439–446.
- Khalid Rafi, H., Starr, T. L., and Stucker, B. E. (2013). A comparison of the tensile, fatigue, and fracture behavior of Ti-6Al-4V and 15-5 PH stainless steel parts made by selective laser melting. *International Journal of Advanced Manufacturing Technology*, 69(5-8):1299–1309.
- Khoshnevis, B. (2004). Automated construction by contour crafting - related robotics and information technologies. *Automation in Construction*, 13(1):5–19.
- Kiyamaz, G. (2005). Strength and stability criteria for thin-walled stainless steel circular hollow section members under bending. *Thin-Walled Structures*, 43(10):1534–1549.
- Kiyamaz, G., Coskun, E., and Cosgun, C. (2007). Behavior and design of seam-welded stainless steel circular hollow section flexural members. *Journal of Structural Engineering ASCE*, 133(12):1792–1800.
- Kobryn, P. A. and Semiatin, S. L. (2001). Mechanical properties of laser-deposited Ti-6Al-4V. In *Proceedings of the Solid Freeform Fabrication Symposium*, pages 6–8, Austin, Texas.
- Kruth, J. P., Mercelis, P., Vaerenbergh, J., Froyen, L., and Rombouts, M. (2005). Binding mechanisms in selective laser sintering and selective laser melting. *Rapid Prototyping Journal*, 11(1):26–36.

- Kuwamura, H. (2003). Local buckling of thin-walled stainless steel members. *Steel Structures*, 3:191–201.
- Lam, D. and Gardner, L. (2008). Structural design of stainless steel concrete filled columns. *Journal of Constructional Steel Research*, 64(11):1275–1282.
- Levy, G. N., Schindel, R., and Kruth, J. P. (2003). Rapid manufacturing and rapid tooling with layer manufacturing (LM) technologies, state of the art and future perspectives. *CIRP Annals - Manufacturing Technology*, 52(2):589–609.
- Liew, A. (2014). *Design of structural steel elements with the continuous strength method*. PhD thesis, Imperial College London.
- Liew, A. and Gardner, L. (2015). Ultimate capacity of structural steel cross-sections under compression, bending and combined loading. *Structures*, 1:2–11.
- Liew, J. Y. R. and Xiong, D. X. (2010). Experimental investigation on tubular columns infilled with ultra-high strength concrete. In Young, B., editor, *Proceedings of the 13th International Symposium on Tubular Structures*, pages 637–645, Hong Kong. Taylor and Francis/Balkema.
- Lim, S., Buswell, R. A., Le, T. T., Austin, S. A., Gibb, A. G. F., and Thorpe, T. (2012). Developments in construction-scale additive manufacturing processes. *Automation in Construction*, 21(1):262–268.
- Liu, Y. and Young, B. (2003). Buckling of stainless steel square hollow section compression members. *Journal of Constructional Steel Research*, 59(2):165–177.
- Luecke, W. E. and Slotwinski, J. A. (2014). Mechanical properties of austenitic stainless steel made by additive manufacturing. *Journal of Research of the National Institute of Standards and Technology*, 119:398–418.
- Lui, W. M., Ashraf, M., and Young, B. (2014). Tests of cold-formed duplex stainless steel SHS beam-columns. *Engineering Structures*, 74:111–121.

- Ma, J., Chan, T. M., and Young, B. (2015). Cold-formed high strength tubular sections of steel grade up to 1100 MPa. In Camotim, D. and Chan, S.-L., editors, *Eighth International Conference on Advances in Steel Structures*, Lisbon.
- Mann, A. P. (1993). The structural use of stainless steel. 71(4):60–69.
- Mann, A. P. (2012). Design and erection of the London Eye and the Wembley National Stadium arch. In Gardner, L., editor, *Tubular Structures - Proceedings of the 14th International Symposium on Tubular Structures*, pages 25–31, London. Taylor & Francis.
- Materialise (2015). Magics.
- Meier, H. and Haberland, C. (2008). Experimental studies on selective laser melting of metallic parts. *Materialwissenschaft und Werkstofftechnik*, 39(9):665–670.
- Mercelis, P. and Kruth, J. P. (2006). Residual stresses in selective laser sintering and selective laser melting. *Rapid Prototyping Journal*, 12(5):254–265.
- Mertens, A., Contrepolis, Q., Dormal, T., Lemaire, O., and Lecomte-Beckers, J. (2012). Ti alloys processed by selective laser melting and by laser cladding: microstructures and mechanical properties. In *12th European Conference on Space Structures, Materials & Environmental Testing*, volume 2012, pages 20–23.
- Mertens, A. I., Reginster, S., Contrepolis, Q., Dormal, T., Lemaire, O., and Lecomte-Beckers, J. (2014). Microstructures and mechanical properties of stainless steel AISI 316L processed by selective laser melting. *Materials Science Forum*, 783-786(August 2015):898–903.
- Mirambell, E. and Real, E. (2000). On the calculation of deflections in structural stainless steel beams: an experimental and numerical investigation. *Journal of Constructional Steel Research*, 54(1):109–133.
- Mower, T. M. and Long, M. J. (2016). Mechanical behavior of additive manufactured, powder-bed laser-fused materials. *Materials Science and Engineering A*, 651:198–213.

- Niendorf, T., Leuders, S., Riemer, A., Richard, H. A., Tröster, T., and Schwarze, D. (2013). Highly anisotropic steel processed by selective laser melting. *Metallurgical and Materials Transactions B: Process Metallurgy and Materials Processing Science*, 44(4):794–796.
- Nseir, J. (2015). *Development of a new design method for the cross-section capacity of steel hollow sections*. PhD thesis, University of Applied Sciences of Western Switzerland, Fribourg.
- Ochi, K. and Choo, B. S. (1993). Ultimate strength and post-buckling behaviour of CHS columns - A comparison between cold-formed and hot-finished sections. In Coutie, M. and Davies, G., editors, *Proceedings of the Fifth International Symposium on Tubular Structures*, pages 163–170, London. E & FN Spon.
- O'Shea, M. D. and Bridge, R. Q. (1997). Local buckling of thin-walled circular steel sections with or without internal restraint. *Journal of Constructional Steel Research*, 41(2):137–157.
- Ozyurt, E. and Wang, Y. C. (2015). Effects of truss behaviour on critical temperatures of welded steel tubular truss members exposed to uniform fire. *Engineering Structures*, 88:225–240.
- Page, C. L. and Anchor, R. D. (1988). Stress corrosion cracking of stainless steels in swimming pool. *The Structural Engineer*, 66(24).
- Paquette, J. A. and Kyriakides, S. (2006). Plastic buckling of tubes under axial compression and internal pressure. *International Journal of Mechanical Sciences*, 48(8):855–867.
- Pearce, J. M., Morris Blair, C., Laciak, K. J., Andrews, R., Nosrat, A., and Zelenika-Zovko, I. (2010). 3-D printing of open source appropriate technologies for self-directed sustainable development. *Journal of Sustainable Development*, 3(4):17.
- Pegna, J. (1997). Exploratory investigation of solid freeform construction. *Automation in Construction*, 5(5):427–437.

- Perkins, I. and Skitmore, M. (2015). Three-dimensional printing in the construction industry: a review. *International Journal of Construction Management*, 15(1):1–9.
- Powell, D. and McCafferty, P. (2012). The United States air force memorial - use of impact dampers to control wind induced vibrations. *The Structural Engineer*, 90(5):14–18.
- Prashanth, K. G., Scudino, S., Klauss, H. J., Surreddi, K. B., Löber, L., Wang, Z., Chaubey, A. K., Kühn, U., and Eckert, J. (2014). Microstructure and mechanical properties of Al-12Si produced by selective laser melting: effect of heat treatment. *Materials Science and Engineering A*, 590(OCTOBER):153–160.
- Prion, H. G. L. and Birkemoe, P. C. (1992). Beam-column behavior of fabricated steel tubular members. *Journal of Structural Engineering ASCE*, 118(5):1213–1232.
- Ramberg, W. and Osgood, W. R. (1943). *Description of stress-strain curves by three parameters*.
- Rasmussen, K. J. R. (2000). Recent research on stainless steel tubular structures. *Journal of Constructional Steel Research*, 54(1):75–88.
- Rasmussen, K. J. R. (2003). Full-range stress-strain curves for stainless steel alloys. *Journal of Constructional Steel Research*, 59(2003):47–61.
- Rasmussen, K. J. R. and Hancock, G. J. (1990). Stainless steel tubular columns - tests and design. In *Tenth international speciality conference on cold-formed steel structures*, pages 471–492, St Louis, Missouri.
- Rasmussen, K. J. R. and Hancock, G. J. (1993a). Design of cold-formed stainless steel tubular members. I: columns. *Journal of Structural Engineering ASCE*, 119(8):2349–2367.
- Rasmussen, K. J. R. and Hancock, G. J. (1993b). Design of cold-formed stainless steel tubular members. II: beams. *Journal of Structural Engineering ASCE*, 119(8):2368–2386.

- Rasmussen, K. J. R. and Rondal, J. (1997). Strength curves for metal columns. *Journal of Structural Engineering ASCE*, 123(6):721–728.
- Rasmussen, K. J. R. and Rondal, J. (2000). Column curves for stainless steel alloys. *Journal of Constructional Steel Research*, 54:89–107.
- Real, E., Arrayago, I., Mirambell, E., and Westeel, R. (2014). Comparative study of analytical expressions for the modelling of stainless steel behaviour. *Thin-Walled Structures*, 83:2–11.
- Reddy, B. D. and Calladine, C. R. (1978). Classical buckling of a thin-walled tube subjected to bending moment and internal pressure. *International Journal of Mechanical Sciences*, 20:641–650.
- Rehme, O. (2009). Selective laser melting of honeycombs with negative Poisson's ratio. *Journal of Laser Micro/Nanoengineering*, 4(2):128–134.
- Ren, Q. X., Han, L. H., Lam, D., and Hou, C. (2014). Experiments on special-shaped CFST stub columns under axial compression. *Journal of Constructional Steel Research*, 98:123–133.
- Rickenbacher, L., Etter, T., Hövel, S., and Wegener, K. (2013). High temperature material properties of IN738LC processed by selective laser melting (SLM) technology. *Rapid Prototyping Journal*, 19(4):282–290.
- Riks, E. (1972). The application of Newton's method to the problem of elastic stability. *Journal of Applied Mechanics*, 39(4):1060–1065.
- Riks, E. (1979). An incremental approach to the solution of snapping and buckling problems. *International Journal of Solids and Structures*, 15(7):529–551.
- Rockey, K. C. and Jenkins, F. (1957). The behaviour of webplates of plate girders subjected to pure bending. *The Structural Engineer*, 35:176–89.
- Rondal, J., Boeraeve, P., Sedlacek, G., and Langenberg, P. (1995). Rotation capacity of hollow beam sections - research project no. 2P. Technical report, CIDECT.

- Rondal, J., Würker, K., Dutta, D., Wardenier, J., and Yeomans, N. (1992). *Structural stability of hollow sections*. Verlag TÜV Rheinland, 1st edition.
- Rossi, B. (2008). *Mechanical properties, residual stresses and structural behavior of thin-walled stainless steel profiles*. PhD thesis, University of Liege.
- Rossi, B., Afshan, S., and Gardner, L. (2013). Strength enhancements in cold-formed structural sections - part II: predictive models. *Journal of Constructional Steel Research*, 83:189–196.
- Rotter, J. M., Sadowski, A. J., and Chen, L. (2014). Nonlinear stability of thin elastic cylinders of different length under global bending. *International Journal of Solids and Structures*, 51(15-16):2826–2839.
- Saab, H. A. and Nethercot, D. A. (1991). Modelling steel frame behaviour under fire conditions. *Engineering Structures*, 13(4):371–382.
- SABS (1997). *Structural use of steel - part 4: the design of cold-formed stainless steel structural members SABS 0162-4:1997*.
- Sakino, K., Nakahara, H., Morino, S., and Nishiyama, I. (2004). Behavior of centrally loaded concrete-filled steel-tube short columns. *Journal of Structural Engineering ASCE*, 130(2):180–188.
- Santorinaios, M., Brooks, W., Sutcliffe, C. J., and Mines, R. A. W. (2006). Crush behaviour of open cellular lattice structures manufactured using selective laser melting. *WIT Transactions on the Built Environment*, 85:481–490.
- Schafer, B. W., Li, Z., and Moen, C. D. (2010). Computational modeling of cold-formed steel. *Thin-Walled Structures*, 48(10-11):752–762.
- Schafer, B. W. and Peköz, T. (1998). Computational modeling of cold-formed steel: characterizing geometric imperfections and residual stresses. *Journal of Constructional Steel Research*, 47(3):193–210.

- Schmidt, H. (1989). Thick-walled tubular members under axial compression. In Niemi, E. and Makelainen, P., editors, *Proceedings of the 3rd International Symposium on Tubular Structures*, pages 356–363, London. Elsevier Applied Science.
- Sedlacek, G., Dahl, W., Stranghöner, N., Kalinowski, B., Rondal, J., and Boeraeve, P. (1995). Investigation of the rotation behaviour of hollow section beams. *ECSC Research project, final report, 7210/SA/119*.
- Seica, M., Packer, J., Ramirez, P., Bell, S. A. H., and Zhao, X. L. (2006). Rehabilitation of tubular members with carbon reinforced polymers. In Packer, J. and Willibald, S., editors, *Proceedings of the 11th International Symposium and IIW International Conference on Tubular Structures*, pages 365–373, London. Taylor and Francis/Balkema.
- Seide, P. and Weingarten, V. (1961). On the buckling of circular cylindrical shells under pure bending. *ASME Journal of Applied Mechanics*, 28(1):112–116.
- Shen, Y., McKown, S., Tsopanos, S., Sutcliffe, C. J., Mines, R. A. W., and Cantwell, W. J. (2010). The mechanical properties of sandwich structures based on metal lattice architectures. *Journal of Sandwich Structures & Materials*, 12(2):159–180.
- Sherman, D. R. (1976). Tests of circular steel tubes in bending. *Journal of the Structural Division ASCE*, 102(11):2181–2195.
- Shu, G., Zheng, B., and Xin, L. (2014). A new design method for stainless steel columns subjected to flexural buckling. *Thin-Walled Structures*, 83:43–51.
- Snyder, M. P., Dunn, J. J., and Gonzalez, E. G. (2013). Effects of microgravity on extrusion based additive manufacturing. In *AIAA SPACE 2013 Conference and Exposition*, pages 1–6.
- Song, B., Zhao, X., Li, S., Han, C., Wei, Q., Wen, S., Liu, J., and Shi, Y. (2015). Differences in microstructure and properties between selective laser melting and traditional manufacturing for fabrication of metal parts: a review. *Frontiers of Mechanical Engineering*, 10(2):111–125.

- SpaceX (2014). SpaceX launches 3D-printed part to space, creates printed engine chamber. <http://www.spacex.com/news/2014/07/31/spacex-launches-3d-printed-part-space-creates-printed-engine-chamber-crewed> [Accessed 10th October 2016].
- Spierings, A., Starr, T., and Wegener, K. (2013). Fatigue performance of additive manufactured metallic parts. *Rapid Prototyping Journal*, 19(2):88–94.
- Spierings, A. B., Herres, N., and Levy, G. (2011). Influence of the particle size distribution on surface quality and mechanical properties in AM steel parts. *Rapid Prototyping Journal*, 17(3):195–202.
- Standards Australia (1998). *AS 4100-1998 steel structures*.
- Standards Australia (2001). *Cold-formed stainless steel structures Australian/ New Zealand Standard AS/NZS 4673:2001*.
- Stangenberg, H. (2000). Report to the ECSC - draft final report ferritic stainless steels. Technical report, RWTH.
- Starossek, U. and Falah, N. (2009). The interaction of steel tube and concrete core in concrete-filled steel tube columns. In Shen, Z., Chen, Y., and Zhao, X., editors, *Proceedings of the 12th International Symposium on Tubular Structures*, pages 75–84, London. Taylor and Francis/Balkema.
- Su, M. N., Young, B., and Gardner, L. (2014a). Deformation-based design of aluminium alloy beams. *Engineering Structures*, 80:339–349.
- Su, M. N., Young, B., and Gardner, L. (2014b). Testing and design of aluminum alloy cross sections in compression. *Journal of Structural Engineering ASCE*, 140(9).
- Su, M. N., Young, B., and Gardner, L. (2016). The continuous strength method for the design of aluminium alloy structural elements. *Engineering Structures*, 122:338–348.
- Su, M. N., Young, B., Gardner, L., and Buchanan, C. A review of flexural design for aluminum alloy circular hollow sections. *Engineering Structures*.

- Sun, J., Peng, Z., Zhou, W., Fuh, J. Y. H., Hong, G. S., and Chiu, A. (2015). A review on 3D printing for customized food fabrication. *Procedia Manufacturing*, 1:308–319.
- Sun, Y., Moroz, A., and Alrbaey, K. (2014). Sliding wear characteristics and corrosion behaviour of selective laser melted 316L stainless steel. *Journal of Materials Engineering and Performance*, 23(2):518–526.
- Sundaram, M. M., Kamaraj, A. B., and Kumar, V. S. (2015). Mask-less electrochemical additive manufacturing: a feasibility study. *Journal of Manufacturing Science and Engineering*, 137(2):021006.
- Symes, M. D., Kitson, P. J., Yan, J., Richmond, C. J., Cooper, G. J. T., Bowman, R. W., Vilbrandt, T., and Cronin, L. (2012). Integrated 3D-printed reactionware for chemical synthesis and analysis. *Nature Chemistry*, 4(5):349–354.
- Talja, A. (1997). Test report on welded I and CHS beams, columns and beam-columns. *Report to ECSC. VTT Building Technology, Finland*.
- Talja, A. and Salmi, P. (1995). Design of stainless steel RHS beams, columns and beam-columns. Technical report, VTT.
- Teng, J. G. and Hu, Y. M. (2007). Behaviour of FRP-jacketed circular steel tubes and cylindrical shells under axial compression. *Construction and Building Materials*, 21(4):827–838.
- The Steel Construction Institute (2006). *Design manual for structural stainless steel*. Third edition.
- The Steel Construction Institute (2017). *Design manual for structural stainless steel*. Fourth edition.
- Theofanous, M., Chan, T. M., and Gardner, L. (2009a). Flexural behaviour of stainless steel oval hollow sections. *Thin-Walled Structures*, 47(6-7):776–787.

- Theofanous, M., Chan, T. M., and Gardner, L. (2009b). Structural response of stainless steel oval hollow section compression members. *Engineering Structures*, 31(4):922–934.
- Theofanous, M. and Gardner, L. (2009). Testing and numerical modelling of lean duplex stainless steel hollow section columns. *Engineering Structures*, 31(12):3047–3058.
- Theofanous, M. and Gardner, L. (2010). Experimental and numerical studies of lean duplex stainless steel beams. *Journal of Constructional Steel Research*, 66(6):816–825.
- Theofanous, M., Saliba, N., Zhao, O., and Gardner, L. (2014). Ultimate response of stainless steel continuous beams. *Thin Walled Structures*, 83:115–127.
- Thornton, C. H., Hungspruke, U., and Joseph, L. M. (1997). Design of the world's tallest buildings - Petronas twin towers at Kuala Lumpur city centre. *The Structural Design of Tall Buildings*, 6:245–262.
- Tibaut, A., Rebolj, D., and Nekrep Perc, M. (2014). Interoperability requirements for automated manufacturing systems in construction. *Journal of Intelligent Manufacturing*, pages 1–12.
- Timoshenko, S. P. and Gere, J. M. (1961). *Theory of elastic stability*. McGraw-Hill Book Company, second edition.
- Toi, Y. and Ine, T. (1988). Basic studies on the crashworthiness of structural elements - part 5: axisymmetric crush tests of circular cylinders and finite element analysis (in Japanese). *Journal of The Society of Naval Architects of Japan*, 164:406–419.
- Toi, Y., Yuge, K., and Obata, K. (1986). Basic studies on the crashworthiness of structural elements - part 2: non-axisymmetric crush tests of circular cylinders and finite element analysis (in Japanese). *Journal of The Society of Naval Architects of Japan*, 161:296–305.
- Tolosa, I., Garciandía, F., Zubiri, F., Zapirain, F., and Esnaola, A. (2010). Study of mechanical properties of AISI 316 stainless steel processed by "selective laser melting",

- following different manufacturing strategies. *International Journal of Advanced Manufacturing Technology*, 51(5-8):639–647.
- Tutuncu, I. and O'Rourke, T. (2006). Compression behavior of nonslender cylindrical steel members with small and large-scale geometric imperfections. *Journal of Structural Engineering ASCE*, 132(8):1234–1241.
- Ushijima, K., Cantwell, W., Mines, R., Tsopanos, S., and Smith, M. (2011). An investigation into the compressive properties of stainless steel micro-lattice structures. *Journal of Sandwich Structures and Materials*, 13:303–329.
- Uy, B., Tao, Z., and Han, L. H. (2011). Behaviour of short and slender concrete-filled stainless steel tubular columns. *Journal of Constructional Steel Research*, 67(3):360–378.
- Van den Berg, G. J. and Van der Merwe, P. (1992). Prediction of corner mechanical properties for stainless steels due to cold forming. In *Eleventh international speciality conference on cold-formed steel structures*, pages 571–586, St. Louis, Missouri, USA.
- Vejrum, T., Bergman, D. W., and Yeung, N. (2006). Stonecutters bridge, Hong Kong: design and construction of the composite upper tower in stainless steel. In *Nordic Steel '09 Construction Conference*, pages 351–358, Malmo, Sweden.
- Vilaro, T., Colin, C., and Bartout, J. D. (2011). As-fabricated and heat-treated microstructures of the Ti-6Al-4V alloy processed by selective laser melting. *Metallurgical and Materials Transactions A: Physical Metallurgy and Materials Science*, 42(10):3190–3199.
- Wardenier, J., Packer, J. A., Zhao, X. L., and Vegte, G. J. V. D. (2011). *Hollow sections in structural applications*. CIDECT, second edition.
- Wei, S., Mau, S. T., Vipulanandan, C., and Mantrala, S. K. (1995). Performance of a new sandwich tube under axial loading: experiment. *Journal of Structural Engineering ASCE*, 121(12):1806–1814.

- Wohlers, T. and Caffrey, T. (2013). Additive manufacturing: going mainstream. *Manufacturing Engineering*, page 5.
- Wohlers, T. and Gornet, T. (2011). History of additive manufacturing. Technical report, Wohlers Associates, inc.
- Wong, K. V. and Hernandez, A. (2012). A review of additive manufacturing. *ISRN Mechanical Engineering*, 2012:1–10.
- Wu, A. S., Brown, D. W., Kumar, M., Gallegos, G. F., and King, W. E. (2014). An experimental investigation into additive manufacturing-induced residual stresses in 316L stainless steel. *Metallurgical and Materials Transactions A: Physical Metallurgy and Materials Science*, 45(13):6260–6270.
- Xiao, Y., He, W., and Choi, K.-K. (2005). Confined concrete-filled tubular columns. *Journal of Structural Engineering ASCE*, 131(3):488–497.
- Yadollahi, A., Shamsaei, N., Thompson, S. M., and Seely, D. W. (2015). Effects of process time interval and heat treatment on the mechanical and microstructural properties of direct laser deposited 316L stainless steel. *Materials Science and Engineering A*, 644:171–183.
- Yadroitsev, I., Thivillon, L., Bertrand, P., and Smurov, I. (2007). Strategy of manufacturing components with designed internal structure by selective laser melting of metallic powder. *Applied Surface Science*, 254(4):980–983.
- Yap, C. Y., Chua, C. K., Dong, Z. L., Liu, Z. H., Zhang, D. Q., Loh, L. E., and Sing, S. L. (2015). Review of selective laser melting: materials and applications. *Applied Physics Reviews*, 2(4).
- Young, B. and Ellobody, E. (2006). Column design of cold-formed stainless steel slender circular hollow sections. *Steel and Composite Structures*, 6(4):285–302.
- Young, B. and Hartono, W. (2002). Compression tests of stainless steel tubular members. *Journal of Structural Engineering ASCE*, 128(6):754–761.

- Young, B. and Liu, Y. (2003). Experimental Investigation of Cold-Formed Stainless Steel Columns. *Journal of Structural Engineering ASCE*, 129:169–176.
- Young, B. and Lui, W.-M. (2005). Behavior of cold-formed high strength stainless steel sections. *Journal of Structural Engineering ASCE*, 131:1738–1745.
- Yu, T. and Teng, J. (2010). Hybrid FRP-concrete-steel double-skin tubular columns with a square outer tube and a circular inner tube: Stub column tests. In Young, B., editor, *Proceedings of the 13th International Symposium on Tubular Structures*, pages 629–636, London. Taylor and Francis/Balkema.
- Yuan, H. X., Wang, Y. Q., Shi, Y. J., and Gardner, L. (2014). Stub column tests on stainless steel built-up sections. *Thin-Walled Structures*, 83:103–114.
- Yuen, R. M. (1992). *Lateral buckling of steel I-section bridge girders braced by U-frames*. PhD thesis, The University of Leeds.
- Zhao, O., Gardner, L., and Young, B. (2015a). Structural performance of stainless steel circular hollow sections under combined axial load and bending - part 1: experiments and numerical modelling. *Thin-Walled Structures*, 101:231–239.
- Zhao, O., Gardner, L., and Young, B. (2015b). Structural performance of stainless steel circular hollow sections under combined axial load and bending - part 2: parametric studies and design. *Thin-Walled Structures*, 101:240–248.
- Zhao, O., Gardner, L., and Young, B. (2016a). Behaviour and design of stainless steel SHS and RHS beam-columns. *Thin-Walled Structures*, 106:330–345.
- Zhao, O., Gardner, L., and Young, B. (2016b). Buckling of ferritic stainless steel members under combined axial compression and bending. *Journal of Constructional Steel Research*, 117:35–48.
- Zhao, O., Gardner, L., and Young, B. (2016c). Testing and numerical modelling of austenitic stainless steel CHS beam-columns. *Engineering Structures*, 111:263–274.

- Zhao, O., Rossi, B., Gardner, L., and Young, B. (2015c). Behaviour of structural stainless steel cross-sections under combined loading - part I: experimental study. *Engineering Structures*, 89:236–246.
- Zhao, O., Rossi, B., Gardner, L., and Young, B. (2015d). Behaviour of structural stainless steel cross-sections under combined loading -part II: numerical modelling and design approach. *Engineering Structures*, 89:247–259.
- Zhao, O., Rossi, B., Gardner, L., and Young, B. (2015e). Experimental and numerical studies of ferritic stainless steel tubular cross sections under combined compression and bending. *Journal of Structural Engineering ASCE*, 142(2).
- Zhao, X. L. (2000). Section capacity of very high strength (VHS) circular tubes under compression. *Thin-Walled Structures*, 37(3):223–240.
- Zhao, X. L., Tong, L. W., and Wang, X. Y. (2010). CFDST stub columns subjected to large deformation axial loading. *Engineering Structures*, 32(3):692–703.
- Zhou, F. and Young, B. (2005). Tests of cold-formed stainless steel tubular flexural members. *Thin-Walled Structures*, 43(9):1325–1337.
- Zhou, F. and Young, B. (2009). Concrete-filled aluminum circular hollow section column tests. *Thin-Walled Structures*, 47(11):1272–1280.
- Zhu, J. and Young, B. (2006a). Aluminum alloy circular hollow section beam-columns. *Thin-Walled Structures*, 44(2):131–140.
- Zhu, J. and Young, B. (2006b). Experimental investigation of aluminum alloy circular hollow section columns. *Engineering Structures*, 28(2):207–215.
- Ziemian, R. (2010). *Guide to Stability Design Criteria for Metal Structures*. John Wiley & Sons, Inc., New York, 6th edition.

Appendix A

ABAQUS Python script

A.1 Overview

Two Python scripts were produced to allow a large number of numerical models in Abaqus CAE to be created and run. The development process is outlined in Section 4.3.8. Example Python scripts for the elastic buckling analysis and the nonlinear analysis are provided herein, the inputted material model within the nonlinear analysis has been reduced in length.

A.2 Elastic buckling analysis

The following code was used for the elastic buckling analysis:

```
#####  
#Define constants , all in mm, N  
CHS_outer_diameter = 25  
CHS_wall_thickness = 2.6  
CHS_mid_thickness_radii = (CHS_outer_diameter/float(2) -  
                           CHS_wall_thickness/float(2))  
Knife_edge_offset = 50
```

```

Eccentricity = 0
Elastic_modulus = 217931
Poissons_ratio = 0.3
Work_directory = r"C:\temp\CB\Python"
CAE_filename = 'C:/temp/CB/Python/101.6x1.5_LA_'
LA_filename = 'LA_101_P_s'
Local_mode = 2
Local_imp = CHS_wall_thickness/float(10)
Global_mode = 1
Global_imp = 0.85
#####

from abaqus import *
from abaqusConstants import *
from caeModules import *
from driverUtils import executeOnCaeStartup
executeOnCaeStartup()
session.viewports['Viewport:_1'].partDisplay.geometryOptions.setValues(
    referenceRepresentation=ON)
Mdb()

session.viewports['Viewport:_1'].setValues(displayedObject=None)
import os

#Work Directory location
os.chdir(Work_directory)

#####
#Define lengths to be modelled
LISTOFLENGTHS = []
for i in range(25, 60, 40):
    LISTOFLENGTHS.append(i)

#####
#Start loop for different lengths

```

```

for length in LISTOFLENGTHS:

    modelname = 'L_'+str(length)
    newmodelnameLA = 'LA_CHS_25_1-5_'+modelname

    Effective_length = length + 2*Knife_edge_offset

    #CAE file location and name
    mdb.saveAs(pathName=newmodelnameLA)

    mdb.Model(name=newmodelnameLA, modelType=STANDARD_EXPLICIT)

    s = mdb.models[newmodelnameLA].ConstrainedSketch(name='__profile__',
        sheetSize=200.0)
    g, v, d, c = s.geometry, s.vertices, s.dimensions, s.constraints
    s.setPrimaryObject(option=STANDALONE)

    #####
    #Inner radius
    s.CircleByCenterPerimeter(center=(0.0, 0.0), point1=(0.0,
        CHS_mid_thickness_radii))
    #####

    p = mdb.models[newmodelnameLA].Part(name=modelname,
        dimensionality=THREE_D, type=DEFORMABLE_BODY)
    p = mdb.models[newmodelnameLA].parts[modelname]

    #####
    #CHS member length
    p.BaseShellExtrude(sketch=s, depth=length)
    #####

    s.unsetPrimaryObject()
    p = mdb.models[newmodelnameLA].parts[modelname]
    session.viewports['Viewport:1'].setValues(displayedObject=p)

```

```

del mdb.models[newmodelnameLA].sketches['_profile_']
p = mdb.models[newmodelnameLA].parts[modelname]
p.DatumPlaneByPrincipalPlane(principalPlane=YZPLANE, offset=0.0)
p = mdb.models[newmodelnameLA].parts[modelname]
f = p.faces
pickedFaces = f.getSequenceFromMask(mask=('[#1_]', ), )
d1 = p.datums
p.PartitionFaceByDatumPlane(datumPlane=d1[2], faces=pickedFaces)
p = mdb.models[newmodelnameLA].parts[modelname]

#####
#Mid-height principle plane location
p.DatumPlaneByPrincipalPlane(principalPlane=XYPLANE,
                              offset=length/float(2))
#####

p = mdb.models[newmodelnameLA].parts[modelname]
f = p.faces
pickedFaces = f.getSequenceFromMask(mask=('[#3_]', ), )
d2 = p.datums
p.PartitionFaceByDatumPlane(datumPlane=d2[4], faces=pickedFaces)

session.viewports['Viewport:1'].partDisplay.setValues(
    sectionAssignments=ON, engineeringFeatures=ON)
session.viewports['Viewport:1'].partDisplay.geometryOptions.setValues(
    referenceRepresentation=OFF)
mdb.models[newmodelnameLA].Material(name='Material-1')

#####
#Elastic material properties
mdb.models[newmodelnameLA].materials['Material-1'].Elastic(table=(
    (Elastic_modulus, Poissons_ratio), ))
#####

#####

```

```

#CHS wall thickness
mdb.models[newmodelnameLA].HomogeneousShellSection(name='Section-1',
    preIntegrate=OFF, material='Material-1', thicknessType=UNIFORM,
    thickness=CHS_wall_thickness, thicknessField='',
    idealization=NO_IDEALIZATION, poissonDefinition=DEFAULT,
    thicknessModulus=None, temperature=GRADIENT,
    useDensity=OFF, integrationRule=SIMPSON, numIntPts=5)
#####

p = mdb.models[newmodelnameLA].parts[modelname]
f = p.faces
faces = f.getSequenceFromMask(mask=('#f_'), )
region = p.Set(faces=faces, name='Set-1')
p = mdb.models[newmodelnameLA].parts[modelname]
p.SectionAssignment(region=region, sectionName='Section-1', offset=0.0,
    offsetType=MIDDLE_SURFACE, offsetField='',
    thicknessAssignment=FROM_SECTION)

a = mdb.models[newmodelnameLA].rootAssembly
session.viewports['Viewport:1'].setValues(displayedObject=a)
session.viewports['Viewport:1'].assemblyDisplay.setValues(
    optimizationTasks=OFF, geometricRestrictions=OFF,
    stopConditions=OFF)
a = mdb.models[newmodelnameLA].rootAssembly
a.DatumCsysByDefault(CARTESIAN)
p = mdb.models[newmodelnameLA].parts[modelname]
a.Instance(name='Part-1-1', part=p, dependent=OFF)

session.viewports['Viewport:1'].assemblyDisplay.setValues(
    adaptiveMeshConstraints=ON)
mdb.models[newmodelnameLA].BuckleStep(name='Step-1', previous='Initial',
    numEigen=30, eigensolver=LANCZOS, minEigen=0.0, blockSize=DEFAULT,
    maxBlocks=DEFAULT)
session.viewports['Viewport:1'].assemblyDisplay.setValues(
    step='Step-1')

```

```

session.viewports[ 'Viewport:_1' ].assemblyDisplay.setValues(
    interactions=ON, constraints=ON, connectors=ON,
    engineeringFeatures=ON, adaptiveMeshConstraints=OFF)
a = mdb.models[newmodelnameLA].rootAssembly

#####
#Bottom BC location
a.ReferencePoint(point=(0.0, -Eccentricity , -Knife_edge_offset))
#####

a = mdb.models[newmodelnameLA].rootAssembly

#####
#Top BC location
a.ReferencePoint(point=(0.0, -Eccentricity , Knife_edge_offset+length))
#####

a = mdb.models[newmodelnameLA].rootAssembly
r1 = a.referencePoints
refPoints1=(r1[5], )
region1=a.Set(referencePoints=refPoints1 , name='m_Set-1')
a = mdb.models[newmodelnameLA].rootAssembly
s1 = a.instances[ 'Part-1-1' ].edges
side1Edges1 = s1.getSequenceFromMask(mask=('[#104_]', ), )
region2=a.Surface(side1Edges=side1Edges1 , name='s_Surf-1')
mdb.models[newmodelnameLA].Coupling(name='Top_coupling' ,
    controlPoint=region1 ,
    surface=region2 , influenceRadius=WHOLE_SURFACE,
    couplingType=KINEMATIC, localCsys=None, u1=ON, u2=ON, u3=ON,
    ur1=ON, ur2=ON, ur3=ON)

a = mdb.models[newmodelnameLA].rootAssembly
r1 = a.referencePoints
refPoints1=(r1[4], )
region1=a.Set(referencePoints=refPoints1 , name='m_Set-2')

```



```

a = mdb.models[newmodelNameLA].rootAssembly
s1 = a.instances['Part-1-1'].edges
side1Edges1 = s1.getSequenceFromMask(mask=('[#240_]', ), )
region2=a.Surface(side1Edges=side1Edges1 , name='s_Surf-2')
mdb.models[newmodelNameLA].Coupling(name='Bottom_coupling',
    controlPoint=region1 , surface=region2 ,
    influenceRadius=WHOLE_SURFACE, couplingType=KINEMATIC,
    localCsys=None, u1=ON, u2=ON, u3=ON, ur1=ON, ur2=ON, ur3=ON)

a = mdb.models[newmodelNameLA].rootAssembly
v1 = a.instances['Part-1-1'].vertices
verts1 = v1.getSequenceFromMask(mask=('[#2_]', ), )
a.Set(vertices=verts1 , name='Mid_point_y_pos')

a = mdb.models[newmodelNameLA].rootAssembly
v1 = a.instances['Part-1-1'].vertices
verts1 = v1.getSequenceFromMask(mask=('[#1_]', ), )
a.Set(vertices=verts1 , name='Mid_point_y_neg')

session.viewports['Viewport:1'].assemblyDisplay.setValues(loads=ON,
    bcs=ON, predefinedFields=ON, interactions=OFF, constraints=OFF,
    engineeringFeatures=OFF)

a = mdb.models[newmodelNameLA].rootAssembly
r1 = a.referencePoints
refPoints1=(r1[5], )

#####
#Applied load
region = a.Set(referencePoints=refPoints1 , name='Set-5')
mdb.models[newmodelNameLA].ConcentratedForce(name='Load-1',
    createStepName='Step-1', region=region , cf3=-1000.0,
    distributionType=UNIFORM, field='', localCsys=None)
#####

a = mdb.models[newmodelNameLA].rootAssembly

```

```

r1 = a.referencePoints
refPoints1=(r1[5], )
region = a.Set(referencePoints=refPoints1, name='Set-6')
mdb.models[newmodelnameLA].DisplacementBC(name='Top_BC',
      createStepName='Step-1', region=region, u1=0.0, u2=0.0, u3=UNSET,
      ur1=UNSET, ur2=0.0, ur3=0.0, amplitude=UNSET,
      buckleCase=PERTURBATION_AND_BUCKLING, fixed=OFF,
      distributionType=UNIFORM, fieldName='', localCsys=None)

a = mdb.models[newmodelnameLA].rootAssembly
r1 = a.referencePoints
refPoints1=(r1[4], )
region = a.Set(referencePoints=refPoints1, name='Set-7')
mdb.models[newmodelnameLA].DisplacementBC(name='Bottom_BC',
      createStepName='Step-1', region=region, u1=0.0, u2=0.0, u3=0.0,
      ur1=UNSET, ur2=0.0, ur3=0.0, amplitude=UNSET,
      buckleCase=PERTURBATION_AND_BUCKLING, fixed=OFF,
      distributionType=UNIFORM, fieldName='', localCsys=None)

session.viewports['Viewport:1'].assemblyDisplay.setValues(mesh=ON,
      loads=OFF, bcs=OFF, predefinedFields=OFF, connectors=OFF)
session.viewports['Viewport:1'].assemblyDisplay.meshOptions.setValues(
      meshTechnique=ON)
a = mdb.models[newmodelnameLA].rootAssembly
partInstances =(a.instances['Part-1-1'], )

#####
#Mesh size
a.seedPartInstance(regions=partInstances, size=CHS_wall_thickness,
      deviationFactor=0.1, minSizeFactor=0.1)
#####

a = mdb.models[newmodelnameLA].rootAssembly
f1 = a.instances['Part-1-1'].faces
pickedRegions = f1.getSequenceFromMask(mask=('[#f_]', ), )

```

```

a.setMeshControls(regions=pickedRegions, elemShape=QUAD)
a = mdb.models[newmodelNameLA].rootAssembly
partInstances = (a.instances['Part-1-1'], )
a.generateMesh(regions=partInstances)

session.viewports['Viewport:1'].assemblyDisplay.setValues(mesh=OFF)
session.viewports['Viewport:1'].assemblyDisplay.meshOptions.setValues(
    meshTechnique=OFF)

import job
mdb.models[newmodelNameLA].keywordBlock.synchVersions(
    storeNodesAndElements=False)
mdb.models[newmodelNameLA].keywordBlock.replace(41, ""
*modal file

**
** BOUNDARY CONDITIONS
**
** Name: Bottom BC Type: Displacement/Rotation "")
mdb.models[newmodelNameLA].keywordBlock.replace(52, ""
*Output, field, variable=PRESELECT

*node file
u "")

mdb.Job(name=newmodelNameLA, model=newmodelNameLA, description='',
        type=ANALYSIS, atTime=None, waitMinutes=0, waitHours=0, queue=None,
        memory=90, memoryUnits=PERCENTAGE,
        getMemoryFromAnalysis=True, explicitPrecision=SINGLE,
        nodalOutputPrecision=SINGLE, echoPrint=OFF, modelPrint=OFF,
        contactPrint=OFF, historyPrint=OFF, userSubroutine='', scratch='',
        multiprocessingMode=DEFAULT, numCpus=1, numGPUs=0)
mdb.save()

#####
#Create input file

```

```

mdb.jobs[newmodelnameLA].writeInput(consistencyChecking=OFF)
#####

#####
#Submit LA job
#mdb.jobs[LA_filename].submit(consistencyChecking=OFF)
#####

```

A.3 Nonlinear analysis

The following script was used for the nonlinear analysis:

```

#####
#Define constants, all in mm, N
CHS_outer_diameter = 25
CHS_wall_thickness = 2.6
CHS_mid_thickness_radial = (CHS_outer_diameter/float(2) -
    CHS_wall_thickness/float(2))
Knife_edge_offset = 50
Eccentricity = 0
Elastic_modulus = 217931
Poissons_ratio = 0.3
Work_directory = r"C:\temp\CB\Python"
CAE_filename = 'C:/temp/CB/Python/101.6x1.5_LA_'
LA_filename = 'LA_101_P_s'
Local_mode = 2
Local_imp = CHS_wall_thickness/float(10)
Global_mode = 1
#####

from abaqus import *
from abaqusConstants import *
from caeModules import *

```

```

from driverUtils import executeOnCaeStartup
executeOnCaeStartup()
session.viewports['Viewport:1'].partDisplay.geometryOptions.setValues(
    referenceRepresentation=ON)
Mdb()

session.viewports['Viewport:1'].setValues(displayedObject=None)
import os

#Work Directory location
os.chdir(Work_directory)

#####
#Define lengths to be modelled
LISTOFLENGTHS = []
for i in range(25, 60, 40):
    LISTOFLENGTHS.append(i)

#####
#Start loop for different lengths
for length in LISTOFLENGTHS:

    modelname = 'L_'+str(length)
    newmodelnameLA = 'LA_CHS_25_1-5_'+modelname
    newmodelnameNL = 'NL_CHS_25_1-5_'+modelname

    #CAE file location and name
    mdb.saveAs(pathName=newmodelnameNL)

    Effective_length = length + 2*Knife_edge_offset
    Global_imp = Effective_length/float(1000)

    mdb.Model(name=newmodelnameNL, modelType=STANDARD_EXPLICIT)

    s = mdb.models[newmodelnameNL].ConstrainedSketch(name='__profile__',

```

```

        sheetSize=200.0)
g, v, d, c = s.geometry, s.vertices, s.dimensions, s.constraints
s.setPrimaryObject(option=STANDALONE)

#####
#Inner radius
s.CircleByCenterPerimeter(center=(0.0, 0.0), point1=(0.0,
        CHS_mid_thickness_radii))
#####

p = mdb.models[newmodelnameNL].Part(name=modelname,
        dimensionality=THREE_D, type=DEFORMABLE_BODY)
p = mdb.models[newmodelnameNL].parts[modelname]

#####
#CHS member length
p.BaseShellExtrude(sketch=s, depth=length)
#####

s.unsetPrimaryObject()
p = mdb.models[newmodelnameNL].parts[modelname]
session.viewports['Viewport:1'].setValues(displayedObject=p)
del mdb.models[newmodelnameNL].sketches['__profile__']

p = mdb.models[newmodelnameNL].parts[modelname]
p.DatumPlaneByPrincipalPlane(principalPlane=YZPLANE, offset=0.0)
p = mdb.models[newmodelnameNL].parts[modelname]
f = p.faces
pickedFaces = f.getSequenceFromMask(mask=('[#1_]', ), )
d1 = p.datums
p.PartitionFaceByDatumPlane(datumPlane=d1[2], faces=pickedFaces)
p = mdb.models[newmodelnameNL].parts[modelname]

#####
#Mid-height principle plane location

```

```

p.DatumPlaneByPrincipalPlane(principalPlane=XYPLANE,
                              offset=length/float(2))
#####

p = mdb.models[newmodelnameNL].parts[modelname]
f = p.faces
pickedFaces = f.getSequenceFromMask(mask=('[#3_]', ), )
d2 = p.datums
p.PartitionFaceByDatumPlane(datumPlane=d2[4], faces=pickedFaces)

session.viewports['Viewport:1'].partDisplay.setValues(
    sectionAssignments=ON, engineeringFeatures=ON)
session.viewports['Viewport:1'].partDisplay.geometryOptions.setValues(
    referenceRepresentation=OFF)
mdb.models[newmodelnameNL].Material(name='Material-1')

#####
#Elastic material properties
mdb.models[newmodelnameNL].materials['Material-1'].Elastic(table=(
    (Elastic_modulus, Poissons_ratio), ))
#####

#####
#CHS wall thickness
mdb.models[newmodelnameNL].HomogeneousShellSection(name='Section-1',
    preIntegrate=OFF, material='Material-1', thicknessType=UNIFORM,
    thickness=CHS_wall_thickness, thicknessField='',
    idealization=NO_IDEALIZATION, poissonDefinition=DEFAULT,
    thicknessModulus=None, temperature=GRADIENT,
    useDensity=OFF, integrationRule=SIMPSON, numIntPts=5)
#####

p = mdb.models[newmodelnameNL].parts[modelname]
f = p.faces
faces = f.getSequenceFromMask(mask=('[#f_]', ), )

```

```

region = p.Set(faces=faces , name='Set-1')
p = mdb.models[newmodelnameNL].parts[modelname]
p.SectionAssignment(region=region , sectionName='Section-1' , offset=0.0,
                    offsetType=MIDDLE_SURFACE, offsetField='',
                    thicknessAssignment=FROM_SECTION)

a = mdb.models[newmodelnameNL].rootAssembly
session.viewports['Viewport:1'].setValues(displayedObject=a)
session.viewports['Viewport:1'].assemblyDisplay.setValues(
    optimizationTasks=OFF, geometricRestrictions=OFF,
    stopConditions=OFF)
a = mdb.models[newmodelnameNL].rootAssembly
a.DatumCsysByDefault(CARTESIAN)
p = mdb.models[newmodelnameNL].parts[modelname]
a.Instance(name='Part-1-1' , part=p, dependent=OFF)

session.viewports['Viewport:1'].assemblyDisplay.setValues(
    adaptiveMeshConstraints=ON)
mdb.models[newmodelnameNL].BuckleStep(name='Step-1' , previous='Initial' ,
    numEigen=20, eigensolver=LANCZOS, minEigen=0.0, blockSize=DEFAULT,
    maxBlocks=DEFAULT)
session.viewports['Viewport:1'].assemblyDisplay.setValues(
    step='Step-1')

session.viewports['Viewport:1'].assemblyDisplay.setValues(
    interactions=ON, constraints=ON, connectors=ON,
    engineeringFeatures=ON, adaptiveMeshConstraints=OFF)
a = mdb.models[newmodelnameNL].rootAssembly

#####
#Bottom BC location
a.ReferencePoint(point=(0.0, -Eccentricity , -Knife_edge_offset))
#####

a = mdb.models[newmodelnameNL].rootAssembly

```



```
#####
```

```
#Top BC location
```

```
a.ReferencePoint(point=(0.0, -Eccentricity, Knife_edge_offset+length))
```

```
#####
```

```
a = mdb.models[newmodelnameNL].rootAssembly
```

```
r1 = a.referencePoints
```

```
refPoints1=(r1[5], )
```

```
region1=a.Set(referencePoints=refPoints1, name='m_Set-1')
```

```
a = mdb.models[newmodelnameNL].rootAssembly
```

```
s1 = a.instances['Part-1-1'].edges
```

```
side1Edges1 = s1.getSequenceFromMask(mask=('[#104_]', ), )
```

```
region2=a.Surface(side1Edges=side1Edges1, name='s_Surf-1')
```

```
mdb.models[newmodelnameNL].Coupling(name='Top_coupling',
```

```
controlPoint=region1, surface=region2, influenceRadius=WHOLE_SURFACE,
```

```
couplingType=KINEMATIC, localCsys=None, u1=ON, u2=ON, u3=ON,
```

```
ur1=ON, ur2=ON, ur3=ON)
```

```
a = mdb.models[newmodelnameNL].rootAssembly
```

```
r1 = a.referencePoints
```

```
refPoints1=(r1[4], )
```

```
region1=a.Set(referencePoints=refPoints1, name='m_Set-2')
```

```
a = mdb.models[newmodelnameNL].rootAssembly
```

```
s1 = a.instances['Part-1-1'].edges
```

```
side1Edges1 = s1.getSequenceFromMask(mask=('[#240_]', ), )
```

```
region2=a.Surface(side1Edges=side1Edges1, name='s_Surf-2')
```

```
mdb.models[newmodelnameNL].Coupling(name='Bottom_coupling',
```

```
controlPoint=region1, surface=region2,
```

```
influenceRadius=WHOLE_SURFACE, couplingType=KINEMATIC,
```

```
localCsys=None, u1=ON, u2=ON, u3=ON, ur1=ON, ur2=ON, ur3=ON)
```

```
a = mdb.models[newmodelnameNL].rootAssembly
```

```
v1 = a.instances['Part-1-1'].vertices
```

```
verts1 = v1.getSequenceFromMask(mask=('[#2_]', ), )
```

```

a.Set(vertices=verts1 , name='Mid_point_y_pos')

a = mdb.models[newmodelnameNL].rootAssembly
v1 = a.instances['Part-1-1'].vertices
verts1 = v1.getSequenceFromMask(mask=('[#1_]', ), )
a.Set(vertices=verts1 , name='Mid_point_y_neg')

session.viewports['Viewport:1'].assemblyDisplay.setValues(loads=ON,
    bcs=ON, predefinedFields=ON, interactions=OFF, constraints=OFF,
    engineeringFeatures=OFF)
a = mdb.models[newmodelnameNL].rootAssembly
r1 = a.referencePoints
refPoints1=(r1[5], )

#####
#Applied load
region = a.Set(referencePoints=refPoints1 , name='Set-5')
mdb.models[newmodelnameNL].ConcentratedForce(name='Load-1',
    createStepName='Step-1', region=region , cf3=-10000.0,
    distributionType=UNIFORM, field='', localCsys=None)
#####

a = mdb.models[newmodelnameNL].rootAssembly
r1 = a.referencePoints
refPoints1=(r1[5], )
region = a.Set(referencePoints=refPoints1 , name='Set-6')
mdb.models[newmodelnameNL].DisplacementBC(name='Top_BC',
    createStepName='Step-1', region=region , u1=0.0, u2=0.0, u3=UNSET,
    ur1=UNSET, ur2=0.0, ur3=0.0, amplitude=UNSET,
    buckleCase=PERTURBATION_AND_BUCKLING, fixed=OFF,
    distributionType=UNIFORM, fieldName='', localCsys=None)

a = mdb.models[newmodelnameNL].rootAssembly
r1 = a.referencePoints
refPoints1=(r1[4], )

```

```

region = a.Set(referencePoints=refPoints1 , name='Set-7')
mdb.models[newmodelnameNL].DisplacementBC(name='Bottom_BC',
      createStepName='Step-1', region=region , u1=0.0, u2=0.0, u3=0.0,
      ur1=UNSET, ur2=0.0, ur3=0.0, amplitude=UNSET,
      buckleCase=PERTURBATION_AND_BUCKLING, fixed=OFF,
      distributionType=UNIFORM, fieldName='', localCsys=None)

session.viewports['Viewport:1'].assemblyDisplay.setValues(mesh=ON,
      loads=OFF, bcs=OFF, predefinedFields=OFF, connectors=OFF)
session.viewports['Viewport:1'].assemblyDisplay.meshOptions.setValues(
      meshTechnique=ON)
a = mdb.models[newmodelnameNL].rootAssembly
partInstances =(a.instances['Part-1-1'], )

#####
#Mesh size
a.seedPartInstance(regions=partInstances , size=CHS_wall_thickness ,
      deviationFactor=0.1, minSizeFactor=0.1)
#####

a = mdb.models[newmodelnameNL].rootAssembly
f1 = a.instances['Part-1-1'].faces
pickedRegions = f1.getSequenceFromMask(mask=('[#f_]', ), )
a.setMeshControls(regions=pickedRegions , elemShape=QUAD)
a = mdb.models[newmodelnameNL].rootAssembly
partInstances =(a.instances['Part-1-1'], )
a.generateMesh(regions=partInstances)

session.viewports['Viewport:1'].assemblyDisplay.setValues(mesh=OFF)
session.viewports['Viewport:1'].assemblyDisplay.meshOptions.setValues(
      meshTechnique=OFF)
import job
mdb.models[newmodelnameNL].keywordBlock.synchVersions(
      storeNodesAndElements=False)
mdb.models[newmodelnameNL].keywordBlock.replace(41, ""

```

**modal file*

**** BOUNDARY CONDITIONS**

**** Name: Bottom BC Type: Displacement/Rotation """)**

mdb.models[newmodelnameNL].keywordBlock.replace(52, ""

***Output, field, variable=PRESELECT**

**node file*

u """)

```
mdb.Job(name=newmodelnameNL, model=newmodelnameNL, description='',
        type=ANALYSIS, atTime=None, waitMinutes=0, waitHours=0, queue=None,
        memory=90, memoryUnits=PERCENTAGE, getMemoryFromAnalysis=True,
        explicitPrecision=SINGLE, nodalOutputPrecision=SINGLE, echoPrint=OFF,
        modelPrint=OFF, contactPrint=OFF, historyPrint=OFF, userSubroutine='',
        scratch='', multiprocessingMode=DEFAULT, numCpus=1, numGPUs=0)
```

```
p = mdb.models[newmodelnameNL].parts[modelname]
```

```
session.viewports['Viewport:1'].setValues(displayedObject=p)
```

#####

#Plastic material model

```
mdb.models[newmodelnameNL].materials['Material-1'].Plastic(table=(
    (70.02249479, 0.0), (71.02314264, 1.46117e-09),
    (72.02379972, 7.35795e-09), (73.02446604, 1.37087e-08),
    (74.0251416, 2.05315e-08), (75.0258264, 2.78447e-08),
    (76.02652045, 3.56673e-08), (77.02722374, 4.40183e-08),
```

#Stress-strain relationship shortened

```
(718.5229942, 0.012870014), (719.6304018, 0.012991546),
(720.7383797, 0.013113494), (721.8469281, 0.013235855),
(722.9560471, 0.013358629), (724.0657369, 0.013481812)))
```

```

a = mdb.models[newmodelnameNL].rootAssembly
session.viewports['Viewport:1'].setValues(displayedObject=a)
session.viewports['Viewport:1'].assemblyDisplay.setValues(
    adaptiveMeshConstraints=ON)
mdb.models[newmodelnameNL].StaticRiksStep(name='Step-1',
    previous='Initial', maintainAttributes=True, minArcInc=1e-36,
    nlgeom=ON)

session.viewports['Viewport:1'].assemblyDisplay.setValues(
    adaptiveMeshConstraints=OFF)
mdb.models[newmodelnameNL].keywordBlock.synchVersions(
    storeNodesAndElements=False)
mdb.models[newmodelnameNL].keywordBlock.setValues(edited = 0)
mdb.models[newmodelnameNL].keywordBlock.synchVersions(
    storeNodesAndElements=False)

mdb.models[newmodelnameNL].keywordBlock.replace(35, """
*imperfection, file="" +newmodelnameLA +""", step=1
"""+str(Global_mode) +""", """+str(Global_imp) +""
"""+str(Local_mode) +""", """+str(Local_imp) +""
**
** MATERIALS
** """)

mdb.Job(name=newmodelnameNL, model=newmodelnameNL, description='',
    type=ANALYSIS, atTime=None, waitMinutes=0, waitHours=0, queue=None,
    memory=90, memoryUnits=PERCENTAGE, getMemoryFromAnalysis=True,
    explicitPrecision=SINGLE, nodalOutputPrecision=SINGLE,
    echoPrint=OFF, modelPrint=OFF, contactPrint=OFF, historyPrint=OFF,
    userSubroutine='', scratch='', multiprocessingMode=DEFAULT,
    numCpus=1, numGPUs=0)
mdb.save()

mdb.models[newmodelnameNL].keywordBlock.synchVersions(

```

```
storeNodesAndElements=False)
```

```
#####
```

```
#Create input file
```

```
mdb.jobs[newmodelnameNL].writeInput(consistencyChecking=OFF)
```

```
#####
```

```
#####
```

```
#Submit NL job
```

```
#mdb.jobs[newmodelnameNL].submit(consistencyChecking=OFF)
```

```
#: The job input file "NL.inp" has been submitted for analysis.
```

```
#####
```
