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**NANYANG
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SINGAPORE

**BEHAVIOUR AND DESIGN OF PRESS-BRAKED
STAINLESS STEEL ANGLE AND CHANNEL
SECTION MEMBERS**

ZHANG LULU

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

2021

Behaviour and Design of Press-braked Stainless Steel Angle and Channel Section Members

Zhang Lulu

School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University

in partial fulfilment of the requirement for the degree of

Doctor of Philosophy

2021

Statement of Originality

I hereby certify that the work embodied in this thesis is the result of original research, is free of plagiarised materials, and has not been submitted for a higher degree to any other University or Institution.

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11/01/2021

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A/Professor Zhao Ou

Authorship Attribution Statement

This thesis contains material from 6 papers published in the following peer-reviewed journals in which I am listed as an author.

Chapter 3 is published as Zhang, L., Tan, K. H. and Zhao, O. Local stability of press-braked stainless steel angle and channel sections: Testing, numerical modelling and design analysis. *Engineering Structures*, **203**, 109869 (2020). DOI: 10.1016/j.engstruct.2019.109869.

The contributions of the co-authors are as follows:

- A/Prof Zhao and Prof Tan provided the initial project direction and bought the test specimens.
- I co-designed the tests with A/Prof Zhao and Prof Tan and performed all the laboratory work at the Construction Technology Lab. I also analyzed the data.
- I performed the numerical modelling and design analyses.
- I prepared the original manuscript draft. The manuscript was revised by A/Prof Zhao and Prof Tan.

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The contributions of the co-authors are as follows:

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- I performed the numerical modelling and design analyses.
- I prepared the original manuscript draft. The manuscript was revised by A/Prof Zhao and Prof Tan.

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ABSTRACT

Growing focus has been put on the use of stainless steel members in constructional engineering, owing to the superior corrosion resistance and durability, high material strength and exceptional ductility, the nature of 100% recyclable and aesthetics of stainless steel compared to conventional carbon steel. Although extensive studies have been conducted on stainless steel structural members with doubly symmetric sections (such as square/rectangular/circular hollow sections and I-sections), there has been rather limited research on the behaviour of mono-symmetric sections (channel sections) and asymmetric sections (angle sections) to date. The focus of this thesis is thus the structural behaviour and design of stainless steel angle and channel section members subject to various loading conditions. A thorough experimental programme was firstly carried out on two press-braked equal-leg angle sections and two press-braked plain channel sections to obtain a test data bank. At the cross-sectional level, 8 stub column tests, 10 laterally restrained beam tests (about the geometric axes for angle sections and minor principal axes for channel sections), 20 combined loading tests (about the major and minor principal axes for channel sections) were conducted to study the local buckling behaviour of angle and channel section under pure compression, pure bending and combined loading, respectively. At the member level, 16 fixed-ended angle section column tests, 12 pin-ended channel section column tests and 10 channel section beam-column tests were conducted to study the flexural-torsional buckling behaviour of angle section columns, flexural buckling behaviour of channel section columns

and global buckling behaviour of channel section beam-columns, respectively. The experimental programme was accompanied by a parallel numerical modelling programme, including validation studies, where finite element models were developed and validated against the test results, and parametric studies, where the validated finite element models were employed to generate further numerical data over a wide range of cross-section dimensions, member effective lengths and initial loading eccentricities. The test and numerical data were then adopted to evaluate the accuracy of the relevant codified design rules and other recently developed design approaches. It was generally found that the design codes lead to conservative cross-section resistance predictions, but scattered member resistance predictions, with both unsafe and conservative results, mainly attributed to the neglect of the pronounced material strain hardening effect and nonlinear material property of stainless steel in the design. Improved design methods were also finally proposed accordingly, offering more accurate and less scattered resistance predictions.

LIST OF PUBLICATIONS

Journal papers

Liang, Y., Jeyapragasam, V. V. K., Zhang, L. and Zhao, O. (2019). Flexural-torsional buckling behaviour of fixed-ended hot-rolled austenitic stainless steel equal-leg angle section columns. *Journal of Constructional Steel Research*, 154, 43–54.

Zhang, L., Tan, K. H. and Zhao, O. (2019a). Experimental and numerical studies of fixed-ended cold-formed stainless steel equal-leg angle section columns. *Engineering Structures*, 184, 134–144.

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Conference papers

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LIST OF SYMBOLS

A	Gross cross-section area
A_{eff}	Effective cross-section area
a_w	Ratio of web area to gross cross-section area
a', b'	Parameters used in DSM-based design approach
B	Outer leg width of angle section or outer flange width of channel section
b	Average ratio of test and numerical to model resistances
C_{mz}	Equivalent uniform moment factor used in SEI/ASCE 8-02 and AS/NZS 4673
C_1, C_2, C_3, C_4	CSM material parameter
c	Flat element width
c_{eff}	Effective plate element width
D_L	Vertical deflection at loading points
D_M	Vertical deflection at mid-span
d_H, d_V	Initial global geometric imperfections about cross-section geometric axes of angle section specimen
d_M, d_m	Initial global geometric imperfections about cross-section principal axes of angle section specimen

d	Distance between two strain gauges
E	Young's modulus
E_{sh}	Strain hardening slope
E_t	Tangent modulus
$E_{0.2}$	Tangent modulus at 0.2% proof stress
e_0	Initial loading eccentricity
e_{0e}	Initial loading eccentricity from cross-section elastic neutral axis
e_{0eff}	Initial loading eccentricity from cross-section effective neutral axis
e_{0p}	Initial loading eccentricity from cross-section plastic neutral axis
F_a	Design flexural buckling stress according to AS/NZS 4673
F_n	Design allowable stress according to SEI/ASCE 8-02
f_{bf}	Elastic critical flexural buckling stress about major principal axis
f_{bt}	Elastic critical torsional buckling stress
f_{cre}	Elastic critical flexural buckling stress about minor principal axis
f_{crft}	Elastic critical flexural-torsional buckling stress
f_{ne}	Design flexural buckling stress about minor principal axis
f_y	Yield stress
G	Shear modulus
H	Outer web width of channel section
I	Second moment of area
I_y	Second moment of area about major principal axis
I_z	Second moment of area about minor principal axis
$k_{d,n}$	Design (ultimate limit state) fractile factor
k_z	EN 1993-1-4 beam-column interaction factor
k_σ	Plate buckling factor
L	Specimen length
L_e	Effective member length
L_f	Flexural span length
L_0	Constant moment span length
l_1, l_2, l_3, l_4	LVDT readings
M	Bending moment

$M_{csm,Rd}$	CSM cross-section bending moment resistance
$M_{csm,y,Rd}$	CSM cross-section bending moment resistance about major axis
$M_{csm,z,Rd}$	CSM cross-section bending moment resistance about minor axis
$M_{EC3,y,Rd}$	EC3 cross-section bending moment resistance about major axis
$M_{EC3,z,Rd}$	EC3 cross-section bending moment resistance about minor axis
M_{eff}	Effective cross-section moment resistance
$M_{eff,y}$	Effective cross-section moment resistance about major axis
$M_{eff,z}$	Effective cross-section moment resistance about minor axis
M_{el}	Cross-section elastic moment resistance
$M_{el,y}$	Cross-section elastic moment resistance about major axis
$M_{el,z}$	Cross-section elastic moment resistance about minor axis
M_{ny}	Cross-section bending resistance about major axis predicted by SEI/ASCE 8-02 and AS/NZS 4673
M_{nz}	Cross-section bending resistance about minor axis predicted by SEI/ASCE 8-02 and AS/NZS 4673
M_{pl}	Cross-section plastic moment resistance
$M_{pl,y}$	Cross-section plastic moment resistance about major axis
$M_{pl,z}$	Cross-section plastic moment resistance about minor axis
M_R	Codified bending moment resistance
$M_{R,csm,y,Rd}$	Reduced CSM cross-section design bending resistance about major axis
M_u	Failure moment
$M_{u,ASCE}$	Failure moment predicted by SEI/ASCE 8-02
$M_{uy,ASCE}$	SEI/ASCE 8-02 design bending moment about major axis
$M_{uz,ASCE}$	SEI/ASCE 8-02 design bending moment about minor axis
$M_{u,AS/NZS}$	Failure moment predicted by AS/NZS 4673
$M_{uy,AS/NZS}$	AS/NZS 4673 design bending moment about major axis
$M_{uz,AS/NZS}$	AS/NZS 4673 design bending moment about minor axis
$M_{uy,EC3}$	EC3 design bending moment about major axis
$M_{uz,EC3}$	EC3 design bending moment about minor axis
$M_{u,csm}$	Failure moment predicted by CSM
$M_{uy,csm}$	CSM design bending moment about major axis

$M_{uz, csm}$	CSM design bending moment about minor axis
$M_{csm, Rd}$	CSM predicted bending moment resistance
$M_{csm, y, Rd}$	CSM predicted bending moment resistance about major axis
$M_{csm, z, Rd}$	CSM predicted bending moment resistance about minor axis
$M_{u, EC3}$	Failure bending moment predicted by EN 1993-1-4
M_{ue}	Failure bending moment with reference to the elastic neutral axis
$M_{u, pred}$	Predicted failure moment
$M_{u, test}$	Test failure moment
$M_{u, FE}$	FE failure moment
$M_{z, Rd}$	EC3 design cross-section bending moment resistance about minor axis
$M_{1st, el, ue}$	Test first-order elastic bending moment at failure load
$M_{2nd, el, ue}$	Test second-order elastic bending moment at failure load
$M_{2nd, iel, ue}$	Test second order inelastic bending moment at failure load
m	Strain hardening coefficient based on $\sigma_{0.2}$ and σ_u
N	Applied compression load
$N_{b, z, Rd}$	EC3 design column minor-axis flexural buckling resistance
$N_{csm, Rd}$	CSM design cross-section compression resistance
N_{cr}	Euler buckling load about minor principal axis
$N_{cr, FT}$	Elastic critical flexural-torsional buckling load
$N_{EC3, Rd}$	EC3 design cross-section compression resistance
N_{eff}	Effective cross-section compression resistance
N_n	Nominal axial compression resistance predicted by SEI/ASCE 8-02 and AS/NZS 4673
N_{pl}	Cross-section yield load
N_R	Codified column buckling resistance
N_u	Failure load
$N_{u, ASCE}$	Failure load predicted by SEI/ASCE 8-02
$N_{u, AS/NZS}$	Failure load predicted by AS/NZS 4673
$N_{u, csm}$	Failure load predicted by CSM
$N_{u, DSM}$	Failure load predicted by DSM-based design approach
$N_{u, EC3}$	Failure load predicted by EN 1993-1-4

$N_{u,FE}$	FE failure load
$N_{u,nft}$	Design pure flexural-torsional buckling resistance
$N_{u,p}$	Failure load predicted by proposed design method
$N_{u,pred}$	Predicted failure load
$N_{u,SCI}$	Failure load predicted by SCI P413
$N_{u,test}$	Test failure load
n	Strain hardening coefficient
$n'_{0.2,1.0}$	Strain hardening coefficient based on $\sigma_{0.2}$ and $\sigma_{1.0}$
R	Cross-section rotation capacity
r	Cross-section internal corner radius
t	Cross-section thickness
V_r	Combined COV incorporating both model and basic variable uncertainties
V_δ	COV of the tests and FE simulations relative to the resistance model
W_{eff}	Effective section modulus
$W_{eff,y}$	Effective section modulus about major axis
$W_{eff,z}$	Effective section modulus about minor axis
W_{el}	Elastic section modulus
$W_{el,y}$	Elastic section modulus about major axis
$W_{el,z}$	Elastic section modulus about minor axis
W_{pl}	Plastic section modulus
$W_{pl,y}$	Plastic section modulus about major axis
$W_{pl,z}$	Plastic section modulus about minor axis
x_3, x_4, y_1, y_2	Distances from LVDTs to corner of angle section
y_c	Distance from outermost compressive fibre to CSM design neutral axis
α	Ratio of width of compressive portion of outstand plate element to total width of outstand plate element
α_{EC3}	Imperfection factor used in EN 1993-1-4
$\alpha_{AS/NZS}, \beta_{AS/NZS}, \lambda_0, \lambda_1$	Parameters used in AS/NZS 4673
α_{csm}	CSM bending parameter
α_{nz}	Magnification factor used in SEI/ASCE 8-02 and AS/NZS 4673

β	Initial torsional geometric imperfection of angle section specimen
β_u	Mid-height torsional rotation at failure load
β_w	Factor related to cross-section class
γ_{M0}	Partial safety factor for cross-section resistance
γ_{M1}	Partial safety factor for member resistance
η	Imperfection factor used in AS/NZS 4673
Δ	Lateral deflection at mid-height of specimen
Δ_{dM}	Mid-height lateral deflection about major principal axis at failure load
Δ_{dm}	Mid-height lateral deflection about minor principal axis at failure load
Δ_f	Parameter used in DSM-based design approach
Δ_u	Mid-height lateral deflection at failure load
δ_u	Axial shortening of specimen at failure load
ε	Strain or EN 1993-1-4 material coefficient
ε_{csm}	CSM predicted failure strain of cross-section
$\varepsilon_{csm,c}$	CSM predicted limiting compressive strain of cross-section
$\varepsilon_{csm,d}$	CSM design strain for the calculation of bending resistance
$\varepsilon_{csm,t}$	CSM predicted limiting tensile strain of cross-section
ε_{max}	Strain gauge values of the maximum compressive fibre
ε_{min}	Strain gauge values of the minimum compressive fibre
ε_u	Strain at ultimate stress
ε_y	Yield strain
$\varepsilon_{0.2}$	Strain at 0.2% proof stress
$\varepsilon_{1.0}$	Strain at 1.0% proof stress
θ	Angle parameter
κ	Curvature of constant moment span of beam specimen
κ_{pl}	Elastic curvature corresponding to M_{pl}
κ_u	Curvature when descending branch of moment–curvature curve drops to M_{pl}
λ_c	Column non-dimensional slenderness for flexural buckling about minor principal axis
λ_{fte}	Column interactive slenderness

$\lambda_{ft}, \bar{\lambda}_{FT}$	Column non-dimensional flexural-torsional buckling slenderness
$\bar{\lambda}_l$	Plate element slenderness
$\bar{\lambda}_p$	Cross-section slenderness
$\bar{\lambda}_z$	Column non-dimensional slenderness about minor principal axis
$\bar{\lambda}_0$	Limiting slenderness
σ	Stress
σ_{cr}	Cross-section elastic critical buckling stress
σ_{csm}	CSM predicted failure stress
σ_u	Ultimate stress
$\sigma_{0.01}$	0.01% proof stress
$\sigma_{0.05}$	0.05% proof stress
$\sigma_{0.2}$	0.2% proof stress
$\sigma_{1.0}$	1.0% proof stress
ϕ	Buckling coefficient used in EN 1993-1-4
$\phi_{AS/NZS}$	Buckling coefficient used in AS/NZS 4673
χ	Reduction factor for column buckling resistance
χ_{FT}	Reduction factor for column flexural-torsional buckling resistance
ω_{dM}	Initial maximum global geometric imperfection amplitude about major principal axis of angle section specimen
ω_{dM}	Initial maximum global geometric imperfection amplitude about minor principal axis of angle section specimen
ω_{f1}, ω_{f2}	Initial maximum local geometric imperfection amplitudes of two legs of angle section specimen or two flanges of channel section specimen
ω_g	Initial mid-height global geometric imperfection amplitude of channel section specimen
ω_w	Initial maximum local geometric imperfection amplitude of web of channel section specimen
ω_β	Initial maximum torsional geometric imperfection amplitude about cross-section shear centre of angle section specimen
ω_0	Initial maximum local geometric imperfection amplitude

CHAPTER 1

INTRODUCTION

1.1 Background

Stainless steel was invented in early 20th century by adding a minimum of 11% chromium into iron alloys, making it highly resistant to rusting and tarnishing, i.e. corrosion-resistant. When stainless steel is exposed to air, chromium interacts with oxygen and a layer of passive and self-repairing chromium oxide (Cr_2O_3) can then be formed at the surface of the base metal, protecting the inner material from further reaction with air and moisture. By adjusting the combinations of different alloying elements including carbon, chromium, nickel, manganese, nitrogen, copper, titanium and molybdenum, different grades of stainless steel can be obtained. Higher material strength and hardness can be achieved through increasing the carbon content. The non-magnetic nature of stainless steel is mainly attributed to the nickel and manganese contents, which are added to stabilise the austenitic microstructure of iron. Nitrogen can improve the tensile properties of stainless steel, making it more ductile. Stainless steels can be commonly grouped into five major families: austenitic, ferritic, duplex,

martensitic and precipitation hardening, and the first three grades are mostly selected for use in construction engineering (Gardner, 2005).

Stainless steel has numerous desirable characteristics as a constructional material, including high strength and ductility, superior resistance against corrosion, aesthetic appearance and the environmental-friendly nature (100% recyclable after use). However, stainless steel has been traditionally regarded as an extravagant solution to structural engineering problems, owing to the high initial material cost, lack of understanding of the structural behaviour and inefficient design rules. A recently developed new grade EN 1.4420 high-chromium austenitic stainless steel has made it possible to lower the material cost without reducing the mechanical properties. Compared to its widely used counterparts such as grade EN 1.4301 and grade EN 1.4404, the new grade EN 1.4420 possesses higher material strength and better resistance against corrosion, due to the higher contents of nitrogen and chromium, but lower material price, owing to the lower molybdenum and nickel contents. The chemical compositions of the new grade 1.4420 and the other two widely used grades 1.4301 and 1.4404 are listed in Table 1.1 (CEN, 2016a), while their nominal mechanical properties and corrosion resistances are given in Table 1.2 (CEN, 2016a), where $\sigma_{0.2}$, $\sigma_{1.0}$ and σ_u are the material 0.2% proof stress, 1.0% proof stress and ultimate stress, respectively, and PREN (pitting resistance equivalent number) is a measure of the resistance of stainless steel against corrosion.

Table 1.1: Chemical compositions of stainless steel grades 1.4301, 1.4404 and 1.4420

Grade	C (%)	Si (%)	Mn (%)	P (%)	S (%)	N (%)	Cr (%)	Mo (%)	Ni (%)
1.4301	≤0.07	≤1.00	≤2.00	0.045	0.015	≤0.10	17.5–19.5	–	8.0–10.5
1.4404	≤0.03	≤1.00	≤2.00	0.045	0.015	≤0.10	16.5–18.5	2.0–2.5	10.0–13.0
1.4420	≤0.07	≤1.00	≤2.00	0.045	0.015	0.14–0.25	19.5–21.5	0.5–1.5	8.0–9.5

Table 1.2: Nominal mechanical properties and corrosion resistances of stainless steel grades 1.4301, 1.4404 and 1.4420

Grade	$\sigma_{0.2}$ (MPa)	$\sigma_{1.0}$ (MPa)	σ_u (MPa)	PREN
1.4301	≥210	≥250	520–720	18
1.4404	≥220	≥260	520–670	24
1.4420	≥320	≥350	630–830	26

1.2 Structural applications

Before 1980s, stainless steel was only used for roofs, facades and minor building components such as prestige cladding, floor plates, wall ties and lintels, due to its high material cost. In the recent decades, in recognition of the material's durability and strength advantages, attractive appearance and low maintenance requirements, stainless steel has been increasingly used to construct main load-bearing structural components in shopping centres, railway and bus stations and airports, with two typical examples illustrated in Figures 1.1 and 1.2. Table 1.3 summarises the structural use of stainless steel in bridge engineering in the recent two decades. More detailed reviews of the structural applications of stainless steel are also given by Baddoo et al. (1997), Gardner (2008a), Gedge (2008), and Baddoo (2008, 2013).

Figure 1.1: The Helix pedestrian bridge in Singapore
(<https://images.app.goo.gl/AYSXm1PCYJvCU3Xb6>)

Table 1.3: Summary of structural use of stainless steel in bridge engineering

No.	Bridge	Country	Type of bridge	Stainless steel component and grade	Date
1	Suransuns Bridge	Switzerland	Stress ribbon pedestrian bridge, 40 m span	Four structural ribbons, 1.4462	1999
2	Millennium Bridge	UK	Tilted box girder arch pedestrian bridge, 80 m span	Arch, 1.4462	2001
3	Apate Bridge	Sweden	Tied beam pedestrian bridge	Main girder, 1.4462	2002
4	Kungalv	Sweden	–	1.4462	2003
5	Puerto Arrupe Bridge	Spain	Box girder pedestrian bridge, length 140 m	Box girder, 1.4362	2003
6	Likholefossen Bridge	Norway	Lightweight pedestrian bridge, 24 m span	All except concrete columns, 1.4162	2004
7	Viaduct Črni Kal	Slovenia	Continuous pre-stressed concrete road bridge	Wind barrier, 1.4162	2004
8	Cala Galdana Bridge	Menorca	Arch road bridge, 45 m main span	Main structure, 1.4462	2005
9	Arco di Malizia	Italy	Single arch road suspension	Arch 1.4362	2005
10	Siena Bridge	Italy	Cable stayed pedestrian bridge, 60 m span	Load bearing structure, 1.4162	2006
11	Piove di Sacco Bridge	Italy	Dual arch road suspension	Arch, deck and casing, 1.4362	2006
12	Celtic Gateway Bridge	Wales	Arch pedestrian bridge, main span 70 m	Load bearing arch, 1.4362	2006
13	Zumaia Bridge	Spain	Pedestrian bridge, length 28 m	428 components and 3 plates, 1.4462	2008
14	The Helix, Marina Bay	Singapore	Tubular pedestrian bridge, length 280 m	Main structure, 1.4462	2009
15	Stockfjarden outlet	Sweden	Road bridge	Load bearing I-section beams 1.4162	2009
16	Meads Reach	UK	Stressed skin arc pedestrian bridge, 55 m span	Stressed skin arc, 1.4462	2009
17	Sant Fruitos Bridge	Spain	Pedestrian arch bridge	Load bearing elements, 1.4162	2009
18	Stonecutters Bridge	China	Cable-stayed road bridge, 1,018 m span	Outer skin of the towers, 1.4462	2009
19	Second Gateway Bridge	Australia	Road bridge over river	Reinforcing bar, 1.4162	2010
20	Harbor Drive Pedestrian Bridge	US	Pedestrian bridge, 162 m curved span	1.4462	2011

Figure 1.2: A stainless steel transmission tower in Bratislava, Slovakia
(<https://unsplash.com/photos/6Au5QdBsGuc>)

The available stainless steel sections in the current market include I-section, tubular (square/rectangular/circular hollow) section, angle section, channel section, Tee section and other irregular sections. For angle and channel sections, they are widely used in the construction engineering due to their simple cross-section shapes and convenience of fabrication and connection. Three methods are generally adopted to fabricate stainless steel angle and channel sections, namely press braking, hot rolling and welding; Figure 1.3 illustrates channel sections fabricated by the three different methods. It is worth noting that press braking is the fastest and most cost-effective fabrication method. Moreover, press braking induces strength enhancements at the corner regions. The present research focuses on the structural behaviour of press-braked angle and channel section members made of the new grade EN 1.4420 high-chromium stainless steel.

Figure 1.3: Press-braked, hot-rolled and laser-welded stainless steel channel sections
(<https://www.montanstahl.com/blog/production-methods-press-brake-hot-roll-laser-weld/>)

1.3 Research objectives

In view of the fact that the initial material cost of stainless steel is roughly three to five times that of normal carbon steel, the primary concern for stainless steel structures is the design efficiency. Press-braked angle and channel section members are used as bracing members in frame-structured buildings to increase the lateral stiffness and as chords in transmission towers to transfer loads, owing to their simple cross-section shapes (enabling easy fabrication and assembly). However, design of angle and channel section members can be rather complicated, mainly because of the torsional-related buckling modes associated with their non-doubly symmetric open cross-section profiles. Therefore, the objectives of this thesis are to investigate the structural behaviour of press-braked high-chromium stainless steel angle and channel section members subject to various loading conditions and to propose efficient design rules for them, underpinned by a comprehensive testing and numerical modelling programme.

1.4 Thesis outline

There are 9 chapters in this thesis, with the corresponding contents briefly summarised as below.

Chapter 1 presents a brief introduction to the background and applications of stainless steel. The research objectives and outline of this thesis are also summarised.

Chapter 2 provides an extensive review of the development of international design standards for stainless steel structures and previous key experimental and numerical investigations relevant to the present research. The research gaps are also highlighted.

Chapter 3 reports an experimental and numerical modelling programme on press-braked high-chromium stainless steel angle and channel section stub columns and beams to study their local buckling behaviour and cross-section resistances. The accuracy of the codified design rules and continuous strength method are assessed based on the experimental and numerical data.

Chapter 4 and Chapter 5 investigate the local buckling behaviour and cross-section resistances of press-braked high-chromium stainless steel channel sections under combined loading. Eccentrically loaded stub column tests about both the major and minor principal axes are firstly conducted, followed by numerical modelling and parametric studies, of which the full details are reported. The interaction curves specified in the design codes are then assessed through comparisons of predicted resistances against the test and numerical failure loads. New design methods are also proposed.

In Chapter 6, the flexural-torsional buckling behaviour and resistances of fixed-ended press-braked high-chromium stainless steel angle section columns are experimentally and numerically investigated. Based on the obtained experimental and numerical data, the accuracy of the codified design provisions and a recently proposed direct strength method for fixed-ended angle section columns failing by flexural-torsional buckling is evaluated. Finally, a new improved design method is developed.

Chapter 7 focuses on the minor-axis flexural buckling behaviour and resistances of press-braked high-chromium stainless steel channel section columns. A testing programme, comprising twelve pin-ended column tests, and a numerical modelling programme, including a validation study and a parametric study, are carried out. On the basis of the experimentally and numerically derived results, the accuracy of the relevant codified design rules is evaluated.

Chapter 8 presents experimental and numerical studies of press-braked high-chromium stainless steel channel section beam-columns. A comprehensive testing programme, in conjunction with a systematic numerical modelling programme, is reported. The codified beam-column design interaction curves are described and then evaluated through comparisons against the experimental and numerical results.

Chapter 9 concludes the key findings of this research. Future research work on stainless steel angle and channel section members are also suggested.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter reviews previous research work that is relevant to this thesis, including development of stainless steel design standards, material properties of stainless steel and experimental, numerical and analytical studies on different types of stainless steel angle and channel section members. Moreover, discussions of the codified design rules and other advanced design methods for stainless steel angle and channel section members are provided. The key findings from previous research are summarised, with the research gaps highlighted.

2.2 Development of design standards for stainless steel structures

The world's first stainless steel design guide, referred to as "Specification for the Design of Light Gauge Cold-Formed Stainless Steel Structural Members", was established mainly based on the research work of Johnson and Winter (1966) and issued by the American Iron

and Steel Institute (AISI) in 1968. A revised version of this specification was later released by AISI in 1974, underpinned by more test results (Wang and Errara, 1971). Then, it was superseded by a new standard published by the American Society of Civil Engineers (ASCE) in 1991, based on an extensive research carried out by Chromium Steels Research Group. The latest specification for the design of cold-formed stainless steel structural members SEI/ASCE 8-02 (ASCE, 2002) was published by ASCE in 2002. For the design of hot-rolled and welded stainless steel structural members, the design guide, titled “Design Guide 27: Structural Stainless Steel” and issued by the American Institute of Steel Construction (AISC) in 2013, should be used.

In Europe, the earliest design guide for stainless steel structures was published by Euro Inox in 1994 and known as “Design Manual for Structural Stainless Steel”. Based on this design guide, the European pre-standard for stainless steel structures, referred to as “ENV 1993-1-4: Design of Steel Structures, Supplementary Rules for Stainless Steels” was then issued in 1996 by the European Standards Organisation (CEN). The current European standard for stainless steel structures EN 1993-1-4 (CEN, 2015) was converted from the pre-standard and issued in 2006. A number of updates of the Euro Inox design manual have been made in the past two decades (Euro Inox, 2002, 2006, 2007) and the most recent edition was released in 2017 by the Steel Construction Institute (SCI, 2017).

The Joint Technical Committee BD-086 of Stainless Steel Structures published the Australian/New Zealand standard for stainless steel “AS/NZS 4673: 2001 Cold-formed stainless steel structures” (AS/NZS, 2001) in 2001. The detailed description of the development of the Australian/New Zealand standard (AS/NZS, 2001) was reported in Rasmussen and Gough (2002). Baddoo (2003) conducted comparisons for stainless steel design standards in Europe, US and Australia/New Zealand.

2.3 Material response of stainless steel

The material behaviour of stainless steel differs from that of the carbon steel, as shown in Figure 2.1. Carbon steel generally exhibits a linear elastic stress–strain response at the initial

stage prior to the attainment of a clearly defined yield point, which is followed by a yield plateau and then some strain hardening, while stainless steel displays a nonlinear stress–strain response with no obvious yield point but a high level of strain hardening and ductility. The proof stress at the 0.2% offset strain $\sigma_{0.2}$ is defined as the material yield stress of stainless steel and used in the design. It is also worth highlighting that stainless steel has exceptionally better ductility than carbon steel, with the corresponding fracture strains for austenitic, duplex and ferritic stainless steels equal to around 40–60%, 30–50% and 15–30%, whereas the fracture strain for carbon steel is considerably small and approximately equal to 20%.

Figure 2.1: Comparisons of typical stress–strain curves of stainless steel and carbon steel

Unlike carbon steel whose stress–strain response can be expressed by an idealised bi-linear material model, stainless steel features nonlinear stress–strain response, which is commonly represented by the Ramberg–Osgood (R–O) material model (Ramberg and Osgood, 1943; Hill, 1944) given by Equation (2.1), in which ε is the material strain, σ is the material stress, E is the Young’s modulus, $\sigma_{0.2}$ is the 0.2% proof stress, n is the strain hardening exponent to reflect the roundness of the stress–strain curve and can be determined by Equations (2.2) or (2.3), in which $\sigma_{0.01}$ and $\sigma_{0.05}$ are the 0.01% proof stress and 0.05% proof stress, respectively. In view of the fact that the R–O material model can only accurately predict the stress–strain

relationship of stainless steel up to the 0.2% proof stress, but overpredict the stresses beyond the 0.2% proof stress, Mirambell and Real (2000) proposed a two-stage R–O material model for the purpose of accurately capturing the full range of stress–strain response. In the two-stage R–O material model, the first stage up to the 0.2% proof stress still adopts the basic R–O expression defined by Equation (2.1), while Equation (2.4) is used for the second stage beyond the 0.2% proof stress until the ultimate stress, in which $E_{0.2}$ is the tangent modulus at the 0.2% proof stress point ($\varepsilon_{0.2}$, $\sigma_{0.2}$) and calculated by Equation (2.5), σ_u is the ultimate stress, $\varepsilon_{0.2}$ and ε_u are the strains corresponding to the 0.2% proof stress and ultimate stress, and m is the strain hardening exponent fitted to the second stage of the stress–strain curve.

$$\varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{\sigma_{0.2}} \right)^n \quad (2.1)$$

$$n = \frac{\ln(20)}{\ln(\sigma_{0.2}/\sigma_{0.01})} \quad (2.2)$$

$$n = \frac{\ln(4)}{\ln(\sigma_{0.2}/\sigma_{0.05})} \quad (2.3)$$

$$\varepsilon = \frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \left(\varepsilon_u - \varepsilon_{0.2} - \frac{\sigma_u - \sigma_{0.2}}{E_{0.2}} \right) \left(\frac{\sigma - \sigma_{0.2}}{\sigma_u - \sigma_{0.2}} \right)^m + \varepsilon_{0.2} \quad \text{for } \sigma_{0.2} < \sigma \leq \sigma_u \quad (2.4)$$

$$E_{0.2} = \frac{E}{1 + 0.002n \frac{E}{\sigma_{0.2}}} \quad (2.5)$$

Rasmussen (2003) further simplified the second stage of the two-stage R–O material model, leading to Equation (2.6), and proposed predictive expressions for σ_u , ε_u and m , as given by Equations (2.7)–(2.9), respectively. These predictive expressions are only related to the three basic parameters used in the original R–O material model, i.e. E , $\sigma_{0.2}$ and n .

$$\varepsilon = \frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \varepsilon_u \left(\frac{\sigma - \sigma_{0.2}}{\sigma_u - \sigma_{0.2}} \right)^m + \varepsilon_{0.2} \quad \text{for } \sigma_{0.2} < \sigma \leq \sigma_u \quad (2.6)$$

$$\frac{\sigma_{0.2}}{\sigma_u} = \begin{cases} 0.2 + 185 \frac{\sigma_{0.2}}{E} & \text{for austenitic and duplex stainless steels} \\ \frac{0.2 + 185 \frac{\sigma_{0.2}}{E}}{1 - 0.0375(n-5)} & \text{for all stainless steel alloys} \end{cases} \quad (2.7)$$

$$\varepsilon_u = 1 - \frac{\sigma_{0.2}}{\sigma_u} \quad (2.8)$$

$$m = 1 + 3.5 \frac{\sigma_{0.2}}{\sigma_u} \quad (2.9)$$

Gardner and Nethercot (2004) proposed that the 1.0% proof stress be adopted to replace the ultimate stress to define the second stage of the two-stage R–O material model, resulting in Equation (2.10), in which $n'_{0.2,1.0}$ is the strain hardening exponent characterising the degree of nonlinearity of the stress–strain curve beyond the 0.2% proof stress and $\varepsilon_{1.0}$ is the strain at the 1.0% proof stress. The stress–strain responses predicted by this model were shown to well agree with the experimental stress–strain curves up to the strains of approximately 10%, demonstrating that this model is suitable to be used for practical design and finite element modelling (Gardner, 2005). For the cases where even larger strains are expected to be attained, for example, in tension members and connections, a three-stage material model proposed by Quach et al. (2008) and Hradil et al. (2013) can be used, enabling accurate representation of the stress–strain curves up to the ultimate strains.

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

Strength enhancements arise in the corner regions of cold-formed (cold-rolled and press-braked) stainless steel cross-sections as a result of plastic deformation during the cold-working process. Rasmussen and Hancock (1993), Gardner (2002), Gardner and Nethercot (2004), Cruise and Gardner (2008a) and Afshan et al. (2013) conducted a series of flat and corner material coupon tests to investigate the strength enhancements in the corner regions of cold-formed stainless steel cross-sections. Based on the measured data, the strength enhancements were generally found to be restricted within the corner regions for press-

braked stainless steel angle and channel sections, but extended beyond the corner regions into the adjacent flat parts by a distance approximately equal to two times the wall thickness for cold-rolled square/rectangular hollow sections (Cruise and Gardner, 2008a). A number of predictive models have also been proposed to predict the strength enhancements in the corner regions of cold-formed stainless steel cross-sections (Gardner and Nethercot, 2004; Ashraf et al., 2005; Cruise and Gardner, 2008a; Rossi et al., 2013).

Through-thickness bending residual stresses are introduced into cold-formed stainless steel cross-sections due to non-uniform through-thickness plastic strains induced during the cold-forming process and can affect their structural behaviour and load-carrying capacities. Measurements of bending residual stresses in cold-formed stainless steel cross-sections were previously conducted (Rasmussen and Hancock, 1993, Cruise and Gardner, 2008b; Jandera et al., 2008; Huang and Young 2012; Zheng et al., 2019). For press-braked stainless steel cross-sections, rather low levels of bending residual stresses were observed in the flat regions, with the maximum value less than 10% of the 0.2% proof stress. In the corner regions, higher bending residual stresses equal to approximately 30% of the 0.2% proof stress were obtained, due to larger localised plastic deformations. However, it should be noted that the bending residual stresses are re-introduced into the coupons during the tensile tests and thus inherently incorporated into the measured stress–strain curves.

2.4 Previous research on angle and channel sections

2.4.1 Angle sections

Equal-leg angle section structural members have been widely used as chords in transmission towers, lateral bracing systems in steel frames and windposts in masonry walls. Despite equal-leg angle section members possessing simple cross-section geometric shapes, their buckling behaviour and design are rather complicated. This can be mainly attributed to the low torsional stiffness of angle sections and thus the relative vulnerability to torsion-related buckling. Given that research on stainless steel angle section members is not extensive, previous experimental, numerical and analytical studies into carbon steel angle section

members are firstly reviewed in this sub-section, followed by a summary of research on stainless steel angle section members. Adluri and Madugula (1996) and Kitipornchai and Lee (1986) conducted pin-ended column tests on hot-rolled carbon steel equal-leg angle sections to investigate their minor-axis flexural buckling behaviour and flexural-torsional buckling behaviour, respectively. Popovic et al. (1999) conducted a series of pin-ended and fixed-ended column tests on cold-formed carbon steel equal-leg angle sections to study their flexural-torsional buckling behaviour and interactive flexural-torsional and minor-axis flexural buckling behaviour. Experimental and numerical investigations into the structural behaviour and load-carrying capacities of pin-ended cold-formed carbon steel equal-leg angle section columns were also carried out by Landesmann et al. (2017), while their fixed-ended counterparts were experimentally and numerically studied by Young (2004) and Ellobody and Young (2005). Young (2005) and Young and Ellobody (2005) reported a testing and numerical modelling programme on fixed-ended cold-formed carbon steel lipped angle section columns and systematically studied their distinct buckling behaviour and failure loads in comparison with those of plain angle section columns. Comparisons of the test and numerical failure loads of carbon steel equal-leg angle section columns against the design load-carrying capacities indicated the undue conservatism of the established design standards (Kitipornchai and Lee, 1986; Adluri and Madugula, 1996; Popovic et al., 1999; Young, 2004, 2005; Ellobody and Young, 2005; Young and Ellobody, 2005; Landesmann et al., 2017). This has prompted the development of improved design approaches for carbon steel equal-leg angle section columns. Specifically, Popovic et al. (2001) proposed that angle section columns be only designed for minor-axis flexural buckling without explicitly accounting for torsional and flexural-torsional buckling. Young (2004) and Young and Ellobody (2005) acknowledged this finding and further proposed an improved design formulation for determining the minor-axis flexural buckling resistances. Rasmussen (2005) developed a simple yet efficient reduction factor to consider the effect of an additional bending moment induced from the shift in effective neutral axis in the design of pin-ended slender angle section columns, in contrast with the current codified design approaches, where lengthy and complicated beam-column interaction expressions are used. Silvestre et al. (2013), Dinis et al. (2012) and Mesacasa Jr. et al. (2014) numerically investigated the buckling and post-buckling behaviour of angle section columns and highlighted (i) the length-dependence of flexural-

torsional buckling and (ii) the existence of interaction from minor-axis flexural buckling. Novel design approaches, which consist of a series of length-dependent strength curves for flexural-torsional buckling and also take due account of the interaction of flexural-torsional and minor-axis flexural buckling, were subsequently developed by Dinis and Camotim (2015) and Landesmann et al. (2017), based on direct strength method (DSM) (Schafer, 2008), and shown to yield a high level of accuracy in predicting the load-carrying capacities for carbon steel equal-leg angle section columns.

Recently, experiments on stainless steel angle section members have also been conducted. Specifically, Sun et al. (2019) performed stub column tests on hot-rolled stainless steel angle sections and studied their cross-section compressive behaviour and resistances, while in-plane bending tests were conducted by Theofanous et al. (2015) and Sun et al. (2019) to investigate the behaviour and resistances of laser-welded and hot-rolled stainless steel angle section beams in bending about the geometric axis. The flexural-torsional buckling behaviour and resistances of fixed-ended hot-rolled stainless steel angle section columns were studied by Liang et al. (2019a), based on laboratory testing and numerical modelling. Sarquis et al. (2020), Sirqueira et al. (2020) and De Menezes et al. (2019) reported a series of tests on fixed-ended hot-rolled stainless steel angle section columns and proposed new design approaches for members failing by flexural-torsional buckling and minor-axis flexural buckling. More recently, Zhang et al. (2020c) experimentally and numerically studied the minor-axis flexural buckling behaviour and resistances of pin-ended hot-rolled stainless steel angle section columns and proposed a new design buckling curve. Table 2.1 summarises the completed tests on hot-rolled and laser-welded stainless steel angle section members. However, it is worth noting that the behaviour of press-braked stainless steel angle section columns remains unexplored and is thus part of the focus of the present thesis.

Table 2.1: Summary of previous tests on stainless steel angle sections

No.	Reference	Material grade	Section type	Specimen type	Boundary condition	No. of tests	Failure mode
1	Theofanous et al. (2015)	1.4307, 1.4571	Laser-welded angle	Beam	Simply-supported	4	LB
2	Sun et al. (2019)	1.4307, 1.4404, 1.4571	Hot-rolled angle	Stub column	Fixed-ended	10	LB
				Beam	Simply-supported	10	LB
3	Liang et al. (2019a)	1.4307, 1.4571	Hot-rolled angle	Column	Fixed-ended	12	FTB
4	De Menezes et al. (2019)	1.4301	Hot-rolled angle	Column	Fixed-ended	13	FTB, FB
5	Sarquis et al. (2020)	1.4301	Hot-rolled angle	Column	Fixed-ended	10	FTB, FB
6	Sirqueira et al. (2020)	1.4301	Hot-rolled angle	Column	Fixed-ended	18	FTB, FB
7	Zhang et al. (2020c)	1.4307, 1.4404, 1.4571	Hot-rolled angle	Column	Pin-ended	12	FB

Note: LB represents local buckling; FTB represents flexural-torsional buckling; FB represents flexural buckling.

Table 2.2: Summary of previous tests on stainless steel channel sections

No.	Reference	Material grade	Section type	Specimen type	Boundary condition	No. of tests	Failure mode
1	Kuwamura (2003)	1.4301, 1.4318	Press-braked channel	Stub column	Fixed-ended	11	LB
			Press-braked lipped channel	Stub column	Fixed-ended	12	LB
2	Theofanous et al. (2015)	1.4307, 1.4404	Laser-welded channel	Beam	Simply-supported	12	LB
3	Dobrić et al. (2017)	1.4301	Press-braked channel	Stub column	Fixed-ended	4	LB
4	Liang et al. (2019c)	1.4307, 1.4404	Laser-welded channel	Stub column	Pin-ended	15	LB
5	Liang et al. (2020)	1.4307, 1.4404	Laser-welded channel	Stub column	Pin-ended	8	LB
6	Lecce and Rasmussen (2006a)	1.4301, 1.4016, 1.4003	Press-braked lipped channel	Column	Fixed-ended	19	DB
7	Becque and Rasmussen (2009a)	1.4301, 1.4016, 1.4003	Press-braked lipped channel	Column	Pin-ended	29	LB+FB
8	Rossi et al. (2010a)	1.4003	Press-braked lipped channel	Column	Fixed-ended	48	DB+FTB

Note: DB represents distortional buckling.

2.4.2 Channel sections

Channel section members have been commonly used in a significant number of structural applications, including lattice structures, façade columns, roof purlins, members of built-up columns, chords in the framing system of high-rise or long-span constructions, members of storage racks and greenhouses. In the past decade, there has been a significant expansion in their applications due to the simple cross-section shapes and ease of fabrication and assembly. While a summary of the previously completed tests on stainless steel channel section members is reported in Table 2.2, a brief review is provided as follows. At cross-sectional level, Kuwamura (2003) and Dobrić et al. (2017) conducted stub column tests on press-braked stainless steel channel sections to study their local buckling behaviour and resistances in compression, while investigations into the in-plane flexural behaviour and resistances of laser-welded stainless steel channel section beams were reported in Theofanous et al. (2015). Liang et al. (2019c, 2019d, 2020) experimentally and numerically studied the structural behaviour and resistances of laser-welded stainless steel channel section stub columns under combined compression and bending. It is worth noting that all the aforementioned studies highlighted the conservatism of the current design codes in predicting cross-section resistances of stainless steel channel sections subjected to isolated and combined loadings, due to the use of the 0.2% proof stress in the design without considering the pronounced strain hardening exhibited by stainless steels. This has prompted the development of an improved continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018; Liang et al., 2019d, 2020), which allows for strain hardening to be considered in the calculation of cross-section resistances of stainless steel channel sections and leads to a much higher degree of design accuracy and consistency over the current design codes. At member level, fixed-ended and pin-ended column tests on press-braked stainless steel lipped channel sections were conducted by Rossi et al. (2010a) and Becque and Rasmussen (2009a) to study their flexural-torsional–distortional interactive buckling behaviour and local–overall interactive buckling behaviour, respectively, while Lecce and Rasmussen (2006a) conducted fixed-ended column tests on press-braked stainless steel lipped channel sections to study their distortional buckling behaviour. More recently, Dobrić et al. (2020) numerically investigated the flexural buckling behaviour and resistances of press-braked stainless steel channel section columns, pointed out the inaccuracy and shortcomings

of the design buckling curves specified in the current design codes and proposed new accurate design buckling curves. It can be seen from the literature review that there have been no experimental studies on (i) the local buckling behaviour of press-braked stainless steel channel section stub columns under combined compression and bending and (ii) the global buckling behaviour of press-braked channel section beam-columns under combined compression and bending, which are thus investigated in this thesis.

2.4.3 Existing design methods

The existing codified design methods for stainless steel angle and channel section members susceptible to local and global buckling are briefly reviewed in this section. Moreover, two advanced design approaches, namely continuous strength method (CSM) and direct strength method (DSM) are also reviewed.

2.4.3.1 Codified design rules for local buckling

The current international design codes follow the traditional cross-section classification concept, together with an ideal elastic-plastic material model, for the calculation of stainless steel cross-section (local buckling) resistances. In the current European code EN 1993-1-4 (CEN, 2015), four classes of cross-section (i.e. Class 1, 2, 3 and 4) are specified, while the American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) divide the cross-sections into non-slender and slender cross-sections. For non-slender (Class 1, 2 and 3) cross-sections, the yield load is specified as the cross-section compression resistance, while the effective compression resistance is used for slender (Class 4) cross-sections. Regarding the cross-section bending resistance, the plastic, elastic and effective moment capacities are respectively adopted for Class 1 or 2, Class 3 and Class 4 cross-sections in EN 1993-1-4 (CEN, 2015), while SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) specify that the cross-section bending resistance of open sections with unstiffened compression plate elements is determined based on initiation of yielding without considering any beneficial plasticity, resulting in the elastic moment capacity for non-slender cross-sections and effective moment capacity for slender cross-

sections. It is worth noting that the effective cross-section compression and bending resistances for slender (Class 4) cross-sections, as calculated from the three design codes employ, are based on the effective width method (Johnson and Winter, 1966). The effective width method treats the cross-section as an assembly of isolated plate elements without taking into account the beneficial interaction effect between plate elements. For the design of channel sections under combined loading, an interaction formula of linear format is used, which is a simple linear summation of the utilisation ratio for each stress resultant, with an upper limit equal to unity. Detailed discussions on the codified local buckling design rules for press-braked stainless steel angle and channel sections are given in Chapters 3–5.

2.4.3.2 Codified treatments of member buckling

For the design of columns susceptible to global buckling (e.g., flexural, torsional and flexural-torsional buckling), the buckling curves, as derived based on the Perry-Robertson buckling formula, are used in EN 1993-1-4 (CEN, 2015). It is worth noting that the effects of initial geometric imperfections and residual stresses on the buckling resistances of columns have been inherently considered in the design buckling curves. SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) employ the same method for column design, in which the tangent modulus of the material stress–strain curve at the design allowable stress is required to be calculated and cumbersome iterations are generally involved. An alternative explicit approach, similar to the method of design buckling curves used in EN 1993-1-4 (CEN, 2015), is also provided in AS/NZS 4673 (AS/NZS, 2001) for columns prone to flexural buckling. With regard to the design of beam-columns, interaction curves are adopted in the three design codes; note that the interaction factor is used to define the shape of the design interaction curve, while the column buckling resistance and cross-section bending resistance are taken as the end points. Detailed discussions on the codified design rules for angle section columns susceptible to flexural-torsional buckling and channel section columns prone to minor-axis flexural buckling are given in Chapters 6 and 7, respectively, while Chapter 8 presents detailed discussions on the codified interaction curves for channel section beam-columns.

2.4.3.3 Continuous strength method (CSM)

The continuous strength method is an advanced deformation-based design method, which replaces the codified cross-section classification concept by a continuous treatment and considers material strain hardening in the calculation of cross-section resistances of stainless steel plated cross-sections. The CSM was originally proposed by Gardner (2002) and Gardner and Nethercot (2004), further modified by Gardner and Ashraf (2006), Ashraf et al. (2006, 2008a) and Gardner (2008b), and finalised by Afshan and Gardner (2013) and Zhao et al. (2017). In the recent decade, the CSM has been further extended to a wider range of applications, including other nonlinear metallic materials like aluminium (Su et al., 2013, 2014, 2016) and hot-rolled and cold-formed carbon steel (Gardner et al., 2011; Liew and Gardner, 2015; Yun and Gardner, 2018a, 2018b; Yun et al., 2018a, 2018b, 2020a, 2020b), as well as a range of structural section types, e.g., square/rectangular hollow sections (Anwar-Ul-Saadat et al., 2016; Liew and Gardner, 2015; Yun and Gardner, 2018a, 2018b; Yun et al., 2018b, 2020a, 2020b; Zhao et al., 2015), circular hollow sections (Ashraf et al., 2008b; Buchanan et al., 2016), I-sections (Liew and Gardner, 2015; Yun et al., 2018a, 2018b) and angle/channel/Tee sections (Zhao and Gardner, 2018; Liang et al., 2019d, 2020). The application of the CSM to the design of press-braked stainless steel angle and channel sections under isolated and combined loadings is further elaborated and discussed in Chapters 3–5.

2.4.3.4 Direct strength method (DSM)

The direct strength method was proposed by Schafer and Peköz (1998a) and allows for more efficient and accurate design of cold-formed steel slender open section members susceptible to local-related buckling (e.g., local buckling and interactive local–global buckling). In comparison with the codified effective width methods which simply treat the cross-section as an assemblage of isolated plate elements without considering the beneficial plate element interaction and usually involve iterations in the calculation of local buckling resistances, the DSM accounts for the beneficial interaction between adjacent plate element and directly relates the local buckling resistance to the cross-section slenderness (Schafer and Ádány, 2006; Schafer, 2008; Ádány and Schafer, 2008). The DSM has been included in the current

North American specification for cold-formed steel structures AISI S100 (AISI S100-16, 2016). Extension of the DSM to the design of stainless steel slender section structural members has also been made (Lecce and Rasmussen, 2006b; Becque et al., 2008; Rossi et al., 2010b; Rossi and Rasmussen, 2012; Huang and Young, 2014). Dinis and Camotim (2015) and Landesmann et al. (2017) also proposed a DSM-based design approach for cold-formed steel angle section columns failing by flexural-torsional buckling. In Chapter 6, the applicability of this DSM-based design approach is firstly assessed for fixed-ended press-braked stainless steel angle section columns, with the advantages and shortcomings highlighted, and a new DSM-based design approach is then proposed.

2.5 Concluding remarks

This chapter has provided an overview of the previous research work that is relevant to this thesis, which includes (i) development of stainless steel design standards, (ii) material properties of stainless steel, (iii) experimental, numerical and analytical studies on different types of stainless steel angle and channel section members and (iv) codified and advanced design methods. It can be concluded from the literature review that (i) the flexural-torsional buckling behaviour of press-braked stainless steel angle section columns and (ii) the local and global buckling behaviour of press-braked stainless steel channel section beam-column elements under combined loading still remain unexplored. Therefore, testing, numerical modelling and design of press-braked stainless steel angle section columns and channel section beam-column elements are the focus of this thesis.

CHAPTER 3

LOCAL BUCKLING BEHAVIOUR OF ANGLE AND CHANNEL SECTIONS UNDER ISOLATED LOADING

3.1 Introduction

Stainless steel is gaining increasing attention in structural engineering for its excellent appearance and mechanical characteristics, including high strength and ductility (Badoo, 2008; Gardner, 2019). Compared to the conventionally used carbon steel, stainless steel also has exceptional resistance against corrosion, leading to significantly reduced regular maintenance work (e.g., spray of anti-corrosion coating) and cost during its service life and therefore life-cycle cost effectiveness. Moreover, non-doubly symmetric section (such as angle section and channel section) members have recently become the research focus, since these members generally exhibit much more complicated buckling behaviour and require much more tedious design procedures than those doubly symmetric section (such as tubular section or I-section) members. With regard to the cross-sectional behaviour of stainless steel angle and channel

sections under isolated loading, Dobrić et al. (2017) conducted four stub column tests on press-braked stainless steel channel sections to study their compression resistances, while laterally restrained beam tests on laser-welded stainless steel plain channel sections were conducted by Theofanous et al. (2015) to study their flexural behaviour and resistances. However, there have been no experimental studies on press-braked stainless steel angle section and channel section beams, and the structural behaviour of press-braked stainless steel angle section and channel section stub columns also requires further experimental investigations.

This chapter reports an experimental and numerical study on press-braked stainless steel angle and channel section stub columns and beams. The laboratory tests are conducted on two equal-leg angle sections and two plain channel sections, and include a total of eight concentrically loaded stub column tests and ten laterally restrained beam tests. The testing programme is supplemented by a numerical modelling programme, where finite element (FE) models are firstly developed to replicate the test structural responses and subsequently employed to carry out parametric studies to obtain further numerical data. The obtained experimental and numerical data are adopted for evaluating the accuracy of the design methods for press-braked stainless steel equal-leg angle and plain channel section members failing by local buckling, as given in the international design standards (CEN, 2015; ASCE, 2002; AS/NZS, 2001) and continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018). The findings of this research have been reported by Zhang et al. (2020a).

3.2 Testing programme

3.2.1 General

A comprehensive testing programme, comprising material testing, initial local geometric imperfection measurements, eight stub column tests under axial compression and ten laterally restrained beam tests, was firstly performed to examine the local buckling behaviour and cross-section resistances of press-braked stainless steel angle and channel sections in compression and bending moment. Two angle sections A 50×50×6 and A 100×100×6 (both with equal legs) and two plain channel sections C 80×50×6 and C 120×60×6, press-braked from the same batch

of 6 mm thick grade EN 1.4420 austenitic stainless steel plates (CEN, 2014b; Liang et al., 2019b; Sun and Zhao, 2019), were used in the experimental investigation. The labelling system for angle sections includes a letter ‘A’ (representing an angle section) and the nominal cross-section sizes (outer leg width $B \times$ outer leg width $B \times$ wall thickness t – see Figure 3.1(a)), while the cross-section designation system for channel sections starts with a letter ‘C’ (representing a channel section), followed by the nominal cross-section sizes (outer web width $H \times$ outer flange width $B \times$ wall thickness t – see Figure 3.1(b)).

Figure 3.1: Notation and locations of coupons

3.2.2 Material tests

Prior to concentrically loaded stub column tests and laterally restrained beam tests, material tensile coupon tests were carried out to obtain the stress–strain curves of the flat and corner parts of each examined angle (or channel) section. A pair of flat coupons and a pair of corner coupons were cut from each section, and the dimensions of the coupons were in conformity with the geometric requirements given in EN ISO 6892-1 (CEN, 2016b) and ASTM E8/E8M-15a (ASTM, 2015), with the widths and gauge lengths equal to 12.5 mm and 50 mm, respectively. A displacement-controlled INSTRON 250 kN hydraulic testing machine was adopted to conduct the tensile coupon tests. The initial loading rate was equal to 0.05 mm/min

until the attainment of the material nominal 0.2% proof stress (320 MPa), after which a faster loading rate of 0.8 mm/min was adopted. Note that the testing machine is equipped with a series of end clamps, with flat and V-shaped clamps respectively adopted for gripping flat and corner coupons. Figure 3.2 shows the test setups for tensile flat and corner coupon tests, where two strain gauges are attached to the coupon at mid-height to measure the tensile strains in the longitudinal direction and an extensometer is mounted onto the necked part of the coupon to record the elongation over the gauge length of 50 mm. The key measured flat and corner material properties for each of the examined angle and channel sections are reported in Table 3.1, where E is the Young's modulus, $\sigma_{0.2}$, $\sigma_{1.0}$ and σ_u are the 0.2% proof stress, 1.0% proof stress and ultimate stress, respectively, $\sigma_u/\sigma_{0.2}$ is the ultimate to 0.2% proof stress ratio (reflecting the level of material strain hardening), ε_u and ε_f are respectively the strains at the ultimate stress and at fracture, and n and $n'_{0.2,1.0}$ are the parameters used in the two-stage Ramberg–Osgood material model (Ramberg and Osgood, 1943; Hill, 1944; Mirambell and Real, 2000; Rasmussen, 2003; Gardner and Ashraf, 2006; Arrayago et al., 2015). Figures 3.3(a)–3.3(d) show the measured material stress–strain curves for the four examined press-braked equal-leg angle and plain channel sections. It can be seen that the corner coupons exhibit higher material strength but lower ductility due to high localised plastic strains induced in the corner regions as a result of cold work.

(a) Flat coupon test setup

(b) Corner coupon test setup

Figure 3.2: Tensile coupon tests setup

Table 3.1: Key material properties measured from the tensile flat and corner coupon tests

(a) Flat coupons

Cross-section	E (GPa)	$\sigma_{0.2}$ (MPa)	σ_u (MPa)	$\sigma_u/\sigma_{0.2}$	ε_u (%)	ε_f (%)	R–O parameters	
							n	$n'_{0.2,1.0}$
A 50×50×6	197.7	272.6	629.8	2.3	43.1	69.0	6.6	1.2
	198.7	283.8	644.2	2.3	47.1	69.0	6.9	1.2
A 100×100×6	192.2	371.8	728.9	2.0	41.9	69.6	9.3	1.0
	199.5	371.9	723.2	1.9	40.6	70.0	7.4	1.2
C 80×50×6	201.5	338.1	703.1	2.1	43.4	70.4	5.9	1.7
	202.8	329.5	689.0	2.1	50.1	70.0	6.1	1.5
C 120×60×6	195.9	305.1	640.0	2.1	45.7	62.0	10.9	1.6
	201.8	285.5	668.9	2.3	45.1	67.0	7.9	3.7

(b) Corner coupons

Cross-section	E (GPa)	$\sigma_{0.2}$ (MPa)	σ_u (MPa)	$\sigma_u/\sigma_{0.2}$	ε_u (%)	ε_f (%)	R–O parameters	
							n	$n'_{0.2,1.0}$
A 50×50×6	182.2	619.5	869.5	1.4	29.7	52.0	2.3	3.6
	192.1	619.3	891.4	1.4	30.6	50.0	2.1	3.7
A 100×100×6	174.8	609.6	820.8	1.3	28.7	51.2	3.5	3.3
	184.0	627.2	845.0	1.3	34.7	50.0	3.7	3.4
C 80×50×6	187.4	549.2	779.2	1.4	37.2	51.0	3.2	3.6
	186.5	554.2	776.1	1.4	28.4	49.6	3.3	3.2
C 120×60×6	187.7	540.8	779.6	1.4	27.3	51.2	2.5	3.3
	189.5	539.3	780.7	1.4	27.2	51.0	2.4	3.4

(a) A 50×50×6

(b) A 100×100×6

(c) C 80×50×6

(d) C 120×60×6

Figure 3.3: Stress–strain curves obtained from material tensile coupon tests

3.2.3 Initial local geometric imperfection measurements

The initial local geometric imperfections of the equal-leg angle section and plain channel section specimens were carefully measured, with the employed test setup (Schafer and Peköz, 1998b) shown in Figure 3.4, where two LVDTs are located at each constituent plate element of the specimen to record the local deviations along two representative longitudinal lines. The initial local imperfection amplitude of each plate element was given as the maximum derivation of the measured data set from a linear regression surface fitted to the data set. Tables 3.2 and 3.3 report the measured initial local geometric imperfection magnitudes of all the constituent plate elements of each specimen, where ω_{f1} and ω_{f2} are the initial local geometric imperfection magnitudes of the two legs of the angle section specimen (or the two flanges of the channel section specimen), while ω_w is initial local geometric imperfection magnitude of the internal web of the channel section specimen. The initial local geometric imperfection magnitude of the specimen ω_0 was defined as the maximum of the initial local geometric imperfection magnitudes measured from all the constituent plate elements.

Figure 3.4: Test setup for initial local geometric imperfection measurements

Table 3.2: Measured geometric dimensions and initial local geometric imperfection magnitudes of press-braked stainless steel equal-leg angle section specimens

Specimen ID	B (mm)	t (mm)	r (mm)	L (mm)	L_f (mm)	L_0 (mm)	ω_{f1} (mm)	ω_{f2} (mm)	ω_0 (mm)
A1-S1	49.98	5.92	8.88	149.0	-	-	0.01	0.02	0.02
A1-S2	49.87	5.90	8.85	149.9	-	-	0.02	0.01	0.02
A2-S1	99.90	5.80	8.70	298.4	-	-	0.01	0.01	0.01
A2-S2	100.00	5.97	8.96	298.2	-	-	0.01	0.04	0.04
A1-B1-L	49.92	5.96	8.94	699.2	599.2	200.0	0.04	0.02	0.04
A1-B2-L	49.92	5.96	8.94	699.2	599.2	200.0	0.04	0.02	0.04
A1-B1-rL	49.78	5.97	8.96	698.7	598.7	200.0	0.03	0.04	0.04
A1-B2-rL	49.78	5.97	8.96	698.7	598.7	200.0	0.03	0.04	0.04
A2-B1-rL	100.02	5.91	8.87	998.2	898.2	300.0	0.02	0.05	0.05
A2-B2-rL	100.02	5.91	8.87	998.2	898.2	300.0	0.02	0.05	0.05

Table 3.3: Measured geometric dimensions and initial local geometric imperfection magnitudes of press-braked stainless steel channel section specimens

Specimen ID	B (mm)	H (mm)	t (mm)	r (mm)	L (mm)	L_f (mm)	L_0 (mm)	ω_{f1} (mm)	ω_{f2} (mm)	ω_w (mm)	ω_0 (mm)
C1-S1	50.13	79.89	5.92	8.88	238.7	-	-	0.01	0.04	0.01	0.04
C1-S2	50.07	79.93	5.94	8.91	239.0	-	-	0.01	0.02	0.02	0.02
C2-S1	60.18	119.80	5.96	8.94	359.8	-	-	0.02	0.02	0.01	0.02
C2-S2	60.41	119.78	5.93	8.90	360.3	-	-	0.01	0.02	0.01	0.02
C1-B-u	50.13	79.89	5.92	8.88	999.1	899.1	300.0	0.01	0.03	0.03	0.03
C1-B-n	50.07	79.93	5.94	8.91	997.3	897.3	300.0	0.02	0.06	0.01	0.06
C2-B-u	60.18	119.80	5.96	8.94	998.6	898.6	300.0	0.02	0.03	0.01	0.03
C2-B-n	60.41	119.78	5.93	8.90	999.2	899.2	300.0	0.03	0.03	0.02	0.03

3.2.4 Concentrically loaded stub column tests

Two repeated stub column tests were carried out on each of the examined press-braked stainless steel angle and channel sections to study its local buckling behaviour and cross-section compression resistance. The nominal specimen lengths were respectively chosen to be three times the outer leg widths and web widths for the angle and channel section stub column specimens, in conformity with the guidelines given in Ziemian (2010). Table 3.2 reports the measured geometric dimensions of each stub column specimen, where L is the specimen length,

B is the outer leg width of angle section or the outer flange width of channel section, H is the outer web width of channel section, t is the wall thickness and r is the corner inner radius. The specimen ID begins with the cross-section identifier (with ‘A1’, ‘A2’, ‘C1’ and ‘C2’ respectively representing A 50×50×6, A 100×100×6, C 80×50×6 and C 120×60×6), followed by a letter ‘S’ (representing a stub column), and ends with a number ‘1’ or ‘2’ (for the purpose of distinguishing two repeated tests), e.g., A1-S1 and A1-S2 represent the two angle section A 50×50×6 stub column specimens used for repeated stub column tests. An INSTRON 2000 kN hydraulic testing machine equipped with fixed bearing plates was adopted to conduct stub column tests at a uniform rate equal to 0.2 mm/min. Figures 3.5(a) and 3.5(b) show the setups for angle and channel section stub column tests, respectively, where two vertical LVDTs are used to record the axial shortening of the specimen and a pair of strain gauges are attached to the mid-height of the specimen to measure the compressive strains in the longitudinal direction. It is worth noting that special anchor devices were used at both ends of the specimens to attain the fixed-ended boundary condition and also preclude local failure at the specimen ends. Specifically, the anchor device for angle section stub columns consists of a big base plate (for seating of the specimens) and three small plates with slotted holes (for clamping the ends of the specimens), as shown in Figure 3.5(a), while the anchor device for channel section stub columns comprises steel stiffeners (inserted between the two flanges) and G-clamps (clamped onto the outer faces of the flanges), as shown in Figure 3.5(b). It is worth noting that the LVDT readings contain both the axial shortenings of the stub column specimen and the deformations of the end platens of the testing machine. The axial shortenings of the stub column specimen were thus obtained by eliminating the deformations of the end platens of the testing machine from the LVDT measurements based on the strain gauge readings (Centre for Advanced Structural Engineering, 1990; Gardner and Nethercot, 2004). This was achieved by assuming that the end platen deformations were proportional to the applied loads and shifting the load–deformation curve obtained from the LVDTs such that its initial slope matched that obtained from the strain gauges. The revised (true) load–axial shortening curves of all the press-braked stainless steel equal-leg angle and plain channel section stub column specimens are shown in Figure 3.6, while the key obtained experimental results are reported in Table 3.4, where $N_{u,test}$ is the failure load, δ_u is the axial shortening of the specimen at the failure load, and $N_{u,test}/A\sigma_{0.2}$ is the ratio of the failure load to the cross-section yield load, where A is the gross cross-section

area. Typical failed press-braked stainless steel angle and channel section stub column specimens A1-S2 and C1-S1 are shown in Figures 3.7 and 3.8, respectively.

Figure 3.5: Stub column test setups

Figure 3.6: Load–axial shortening curves of the tested stub column specimens

Table 3.4: Summary of stub column test results

Specimen ID	Cross-section class*	$N_{u,test}$ (kN)	δ_u (mm)	$N_{u,test}/A\sigma_{0.2}$
A1-S1	3	244.5	2.13	1.31
A1-S2	3	246.5	2.79	1.33
A2-S1	4	358.2	0.96	0.82
A2-S2	4	356.5	0.94	0.80
C1-S1	3	440.0	5.05	1.21
C1-S2	3	437.9	5.02	1.20
C2-S1	3	548.6	4.62	1.13
C2-S2	3	549.0	4.24	1.13

* The cross-section classes are defined according to EN 1993-1-4 (CEN, 2015)

Figure 3.7: Experimental and numerical failure modes for press-braked stainless steel equal-leg angle section stub column specimen A1-S2

Figure 3.8: Experimental and numerical failure modes for press-braked stainless steel plain channel section stub column specimen C1-S1

3.2.5 Laterally restrained beam tests

Ten laterally restrained beam tests were conducted on the four press-braked stainless steel equal-leg angle and plain channel sections in the four-point bending configuration, to investigate their in-plane flexural responses and resistances under constant bending moment. For each channel section, two beam tests were performed about the cross-section minor principal axes in both the ‘u’ and ‘n’ orientations; note that ‘u’ and ‘n’ orientation bending (see Figures 3.9(a) and 3.9(b)) induces the maximum tensile stresses in the internal web and flange tip, respectively. The setup for channel section beam tests is shown in Figure 3.10, where the beam specimen is placed on two steel rollers, which are located at a distance of 50 mm from the specimen ends and provide simply-supported boundary condition about the axis of bending, and loaded at third-points of its flexural span (i.e. the span between the two end steel rollers) through a hardened spreader beam, 70 mm thick steel stiffeners are bolted to the beam specimen at the supports and loading points to avoid local bearing failure at these positions, and three line transducers are placed at the mid-span and loading points to monitor the vertical deflections of the beam specimen at the three locations during testing. For equal-leg angle sections, beam tests were performed about their cross-section geometric axes (parallel to the angle legs), and there are also two bending orientations – ‘L’ and ‘reverse L’ – associated with equal-leg angle section beams bent about the geometric axes, as shown in Figures 3.9(c) and 3.9(d), where bending in the ‘L’ and ‘reverse L’ orientations induces the maximum tensile and compressive stresses in the horizontal legs, respectively. For each of the two studied equal-leg angle sections, ‘reverse L’ orientation bending tests were conducted, while additional beam tests were also performed on angle section A 50×50×6 in the ‘L’ orientation. Note that for each bending case, two nominally identical angle section beams were paired up, bolted to the same set of 70 mm thick spacer plates at the two loading points and two supports (see Figure 3.11), and tested together. The spacer plates were used to (i) strengthen the cross-sections at the loading points and supports to avoid local bearing failure, (ii) form compound solid sections at these positions to preclude the possibility of lateral-torsional buckling of the beam specimens, and (iii) facilitate the application of vertical loads without introducing additional torsion. The four-point bending test setup for angle section beams is the same as that used for channel section beams shown in Figure 3.10. The beam tests were all performed at a uniform rate of 1 mm/min. The geometric dimensions of the press-braked stainless steel angle and channel

section beam specimens are respectively listed in Table 3.2 and Table 3.3, where L_f is the length of the flexural span and L_0 is the constant moment span (i.e. the span between the two loading points). The specimen ID starts with the cross-section identifier, followed by a letter ‘B’ (representing a beam) and a number ‘1’ or ‘2’ (identifying the two angle section beams tested in pair), and ends with the bending orientation, e.g., A2-B1-rL and C1-B-u.

(a) Channel section bent in the ‘u’ orientation

(b) Channel section bent in the ‘n’ orientation

(c) Angle section bent in the ‘L’ orientation

(d) Angle section bent in the ‘reverse L’ orientation

Figure 3.9: Elastic neutral axes, plastic neutral axes and bending orientations for angle and channel sections

Figure 3.10: Four-point bending test setup

Figure 3.11: Schematic diagram of paired angle section beams

Figures 3.12 and 3.13 show typical deformed angle and channel section beam specimens A2-B1-rL and C1-B-u upon testing, respectively, exhibiting in-plane flexural failure. The moment–curvature curves obtained from the laterally restrained beam tests on press-braked stainless steel equal-leg angle and plain channel sections are shown in Figure 3.14, where κ is the curvature of the constant moment span of the beam specimen and can be calculated from Equation (3.1) (Chan and Gardner, 2008), where D_L and D_M are the measured vertical deflections at the loading points and mid-span, respectively. The key experimental results are presented in Table 3.5, where $M_{u,test}$ is the failure bending moment, $M_{u,test}/M_{pl}$ and $M_{u,test}/M_{el}$ are the ratios of the failure bending moment to the cross-section plastic and elastic moment capacities, respectively, where $M_{pl}=W_{pl}\sigma_{0.2}$ and $M_{el}=W_{el}\sigma_{0.2}$ are given as the products of the material 0.2% proof stress and the plastic and elastic section moduli, respectively; note that for each channel section, W_{el} and W_{pl} are determined about the elastic neutral axis (ENA) and plastic neutral axis (PNA) along the minor principal axis direction (see Figure 3.9), while for each angle section, W_{el} and W_{pl} are calculated with respect to the ENA and PNA along the geometric axis direction (see Figure 3.9), and $R=\kappa_u/\kappa_{pl}-1$ is the rotation capacity, where κ_u is defined as the curvature at which the descending branch of the moment–curvature curve drops to M_{pl} and κ_{pl} is equal to M_{pl}/EI , and I is the second moment of area. Note that the experimental moment–curvature curves of some beam specimens show a rising trend or become flat even at unrealistically large curvatures, but the tests must be stopped due to safety concerns. For these cases, the failure bending moments and rotation capacities are annotated with a ‘>’ sign in Table 3.5.

$$\kappa = \frac{8(D_M - D_L)}{4(D_M - D_L)^2 + L_0^2} \quad (3.1)$$

Figure 3.12: Experimental and numerical failure modes for press-braked stainless steel equal-leg angle section beam specimen A2-B1-rL

Figure 3.13: Experimental and numerical failure modes for press-braked stainless steel plain channel section beam specimen C1-B-u

Figure 3.14: Moment–curvature curves of the tested beam specimens

Table 3.5: Summary of four-point bending test results

Specimen ID	Cross-section class*	$M_{u,test}$ (kNm)	$M_{u,test}/M_{el}$	$M_{u,test}/M_{pl}$	κ_{pl} (m ⁻¹)	κ_u (m ⁻¹)	R
A1-B1-L	2	2.49	1.95	1.01	0.10	1.35	12.47
A1-B2-L	2	2.49	1.95	1.01	0.10	1.35	12.47
A1-B1-rL	3	>2.54	2.00	1.04	0.10	>1.24	>11.24
A1-B2-rL	3	>2.54	2.00	1.04	0.10	>1.24	>11.24
A2-B1-rL	4	9.99	2.05	1.09	0.04	>0.65	>14.48
A2-B2-rL	4	9.99	2.05	1.09	0.04	>0.65	>14.48
C1-B-u	1	5.44	2.14	1.18	0.11	>1.09	>9.09
C1-B-n	1	>5.45	2.14	1.18	0.11	>1.17	>9.88
C2-B-u	3	7.41	2.17	1.18	0.07	>0.90	>11.51
C2-B-n	1	7.87	2.30	1.25	0.07	>1.25	>16.43

* The cross-section classes are defined according to EN 1993-1-4 (CEN, 2015).

3.3 Numerical modelling programme

3.3.1 General

A numerical modelling investigation was performed using the finite element analysis package ABAQUS (ABAQUS, 2012) to supplement the experimental study. FE models were initially developed to replicate the test structural responses of press-braked stainless steel equal-leg angle and plain channel section stub columns and laterally restrained beams, and afterwards employed to carry out parametric studies to generate further numerical data over a wide range of cross-section dimensions. The obtained numerical data were used, together with the test results, to evaluate the accuracy of the local buckling design rules for press-braked stainless steel angle and channel section structural components specified in the established design standards (CEN, 2015; ASCE, 2002; AS/NZS, 2001) and continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018).

3.3.2 Development of FE models

The 4-noded shell element with reduced integration, S4R, has been extensively used in the numerical simulations of thin-walled metallic (stainless steel (Becque and Rasmussen, 2009b; Dobrić et al., 2017, 2020; Liang et al., 2019a, 2019c, 2019d, 2020; Sun and Zhao, 2019; Saliba and Gardner, 2013), carbon steel (Xiang et al., 2020; Wang et al., 2020a, 2020b; Zhang et al., 2019b, 2020b, 2020d) and aluminium alloy (Zhu et al., 2019; Zhou and Young, 2020)) structural members of open and tubular sections, and was therefore also adopted herein. The size of the adopted S4R element was selected to be the same as the wall thickness of the modelled cross-section t , based on a prior mesh sensitivity study examining a series of mesh sizes ranging from $0.2t$ to $3t$. For each press-braked stainless steel angle or channel section, the measured (engineering) flat and corner stress–strain curves were firstly converted into the respective true stress–true plastic strain curves, and then assigned to the flat and corner parts of the corresponding angle or channel section FE model. Rasmussen and Hancock (1993) and Cruise and Gardner (2008b) have conducted membrane and bending residual stress measurements on cold-formed stainless steel sections and concluded that the magnitude of the membrane residual stresses was very small compared to that of the bending residual stresses

and thus the influence of the membrane residual stresses on the behaviour of cold-formed stainless steel structural members was negligible. The bending residual stresses, which were evidenced by the longitudinal curvature of the tensile coupons when they were extracted from the cold-formed stainless steel sections, were approximately reintroduced during tensile testing as the coupons were returned to their straight configuration under the application of tensile loading (Rasmussen and Hancock, 1993; Cruise and Gardner, 2008b). Therefore, the effect of the bending residual stresses is considered to be inherently presented into the measured material stress–strain responses (Rasmussen and Hancock, 1993; Cruise and Gardner, 2008b). On this basis, and coupled with the fact that the studied local buckling behaviour is generally insensitive to residual stresses, explicit measurements and modelling of both membrane and bending residual stresses in press-braked (cold-formed) stainless steel angle and channel section stub columns or beams were thus deemed unnecessary. Suitable boundary conditions were defined to replicate the fixed-ended and simply-supported boundary conditions used in the concentrically loaded stub column and laterally restrained beam tests, respectively. For each stub column FE model, the end sections were coupled with two concentric reference points, through which the fixed-ended boundary condition was defined: one reference point was fully restrained and the other one was only allowed to translate in the longitudinal direction. For each beam FE model, the two end sections of the flexural span were respectively coupled with the bottom edge lines (or points) of the cross-sections, with one line (or point) allowed for rotation about the bending axis as well as longitudinal translation and the other line (or point) allowed only for rotation about the bending axis, to attain the same simply-supported boundary condition adopted in the experiments. In addition, each cross-section at the loading point was coupled with its top edge line (or point), allowed to have rotation about the bending axis as well as vertical and longitudinal translations; this replicated the four-point bending configuration employed in the experiments. Initial local geometric imperfections were included into the stub column and laterally restrained beam FE models in the form of the respective lowest elastic buckling mode shapes. Three imperfection magnitudes – the measured maximum local imperfection values ω_0 and 1/10 and 1/100 of the wall thicknesses of the cross-sections – were used to scale the distribution patterns of the initial local geometric imperfections, to explore the sensitivity of the developed angle and channel section stub column and laterally restrained beam FE models to the imperfection magnitudes.

3.3.3 Validation of FE models

Upon development of the FE models, nonlinear static analysis was carried out to derive the failure loads (or bending moments), load–axial shortening (or moment–curvature) curves and failure modes, which were then compared with the test results, enabling the accuracy of the developed FE models to be assessed. The ratios of the numerically obtained failure loads (i.e. compressive loads for stub columns and bending moments for beams) to the experimentally obtained failure loads are reported in Tables 3.6 and 3.7. It was generally found that incorporation of the imperfection magnitudes of $t/100$ and ω_0 into the FE models leads to accurate predictions of the test failure loads, while the use of the imperfection magnitudes of $t/10$ results in slightly conservative failure load predictions. Comparisons between the experimental and numerical load–axial shortening curves for typical press-braked stainless steel equal-leg angle and plain channel section stub column specimens A1-S2 and C1-S1 are shown in Figure 3.15, while Figure 3.16 shows comparisons between the test and FE moment–curvature curves for typical angle and channel section beam specimens A2-B1-rL and C1-B-u, all indicating good agreement. The failure modes displayed by typical angle and channel section stub column and beam specimens were also found to be well replicated by their numerical counterparts, as shown in Figures 3.7, 3.8, 3.12 and 3.13. Therefore, it can be concluded that the developed FE models can accurately replicate the experimental structural behaviour of press-braked stainless steel equal-leg angle and plain channel section stub columns and beams, and are therefore considered to be validated.

Table 3.6: Comparisons of stub column test failure loads with numerical failure loads for various initial local geometric imperfection magnitudes

Specimen ID	$N_{u,FE}/N_{u,test}$		
	ω_0	$t/10$	$t/100$
A1-S1	1.02	0.88	0.98
A1-S2	1.01	0.87	0.97
A2-S1	1.00	0.84	0.96
A2-S2	0.97	0.85	0.96
C1-S1	1.03	0.91	1.01
C1-S2	1.03	0.92	1.01
C2-S1	1.00	0.90	0.98
C2-S2	1.00	0.90	0.98
Mean	1.01	0.88	0.98
COV	0.02	0.03	0.02

Table 3.7: Comparisons of four-point bending test failure moments with numerical failure moments for various initial local geometric imperfection magnitudes

Specimen ID	$M_{u,FE}/M_{u,test}$		
	ω_0	$t/10$	$t/100$
A1-B1-L	0.90	0.90	0.90
A1-B2-L	0.90	0.90	0.90
A1-B1-rL	1.01	1.01	1.01
A1-B2-rL	1.01	1.01	1.01
A2-B1-rL	1.05	1.05	1.05
A2-B2-rL	1.05	1.05	1.05
C1-B-u	0.97	0.91	0.97
C1-B-n	1.00	1.00	1.00
C2-B-u	0.92	0.84	0.92
C2-B-n	0.95	0.95	0.95
Mean	0.97	0.95	0.97
COV	0.05	0.08	0.06

Figure 3.15: Experimental and numerical load–axial shortening curves for stub column specimens A1-S2 and C1-S1

Figure 3.16: Experimental and numerical moment–curvature curves for beam specimens A2-B1-rL and C1-B-u

3.3.4 Parametric studies

In this section, the validated FE models were used to conduct parametric studies to generate further numerical data over a wide range of cross-section dimensions. Specifically, for the modelled equal-leg angle sections, four leg widths of 70 mm, 90 mm, 110 mm and 130 mm were selected, while for the modelled plain channel sections, the outer web widths were fixed at 150 mm, with the outer flange widths ranging from 75 mm to 150 mm, resulting in a wide spectrum of cross-section aspect ratios from 1.0 to 2.0; moreover, the wall thicknesses were varied from 2 mm to 28 mm, to cover both non-slender and slender equal-leg angle and plain channel sections. For each stub column FE model, the length was taken as three times the mean outer dimension of the modelled cross-section, while for each beam FE model, the length of the flexural span was kept at 1000 mm, with two concentrated loads applied to the central 300 mm of the flexural span. It is also worth noting that the measured flat and corner material properties of channel section C 80×50×6 were used in the present parametric studies and the initial local geometric imperfection amplitudes of $t/100$ were adopted for all the FE models. In total, 123 and 103 numerical parametric study results were obtained for press-braked stainless steel equal-leg angle section (and plain channel section) stub columns and beams, respectively.

3.4 Assessment of international design standards

3.4.1 EN 1993-1-4 (EC3)

3.4.1.1 General

The current European code EN 1993-1-4 (CEN, 2015) follows the traditional cross-section classification concept, together with an ideal elastic-plastic material model, for the calculation of cross-section (local buckling) resistances of stainless steel components. The classification of a cross-section is made according to its most slender plate element, while each plate element is classified by comparing its flat width-to-thickness ratio (c/t) with the Eurocode slenderness limits, which are dependent on the material characteristics, the plate element type and the stress distribution throughout the plate width; note that c is taken as the flat element widths excluding the corner radii for internal webs of channel sections, but given as the outer element widths for

outstand flanges of channel sections or legs of angle sections. A total of four classes of cross-sections are specified in EN 1993-1-4 (CEN, 2015): (i) cross-sections defined as Class 1 and Class 2 can attain the yield loads ($A\sigma_{0.2}$) under compression and the plastic moment resistances ($W_{pl}\sigma_{0.2}$) in bending; (ii) Class 3 sections are still capable of achieving the yield loads ($A\sigma_{0.2}$) under compression, but with the bending moment resistances limited to the elastic moment resistances ($W_{el}\sigma_{0.2}$); (iii) Class 4 sections fail before the 0.2% proof stress $\sigma_{0.2}$ is attained, limiting the cross-section load-carrying resistances to the effective compression and bending moment resistances ($A_{eff}\sigma_{0.2}$ and $W_{eff}\sigma_{0.2}$), where A_{eff} and W_{eff} are respectively the cross-section effective area and modulus, and determined based on the effective width formulation given by Equation (3.2), where c_{eff} is the effective plate element width and $\bar{\lambda}_l$ is the plate element slenderness, as obtained from Equation (3.3), where $\varepsilon=[(235/\sigma_{0.2})(E/210000)]^{0.5}$ is a material coefficient, and k_σ is the buckling factor depending on the plate element type and the stress distribution throughout the plate width, and can be obtained according to Clause 4.4 of EN 1994-1-5 (CEN, 2006). In the following Section 3.4.1.2, the accuracy of the slenderness limits for classification of outstand and internal plate elements of press-braked stainless steel angle and channel sections was assessed based on the test and numerical data, while assessment of the EC3 cross-section compression and bending moment resistance predictions are reported in Section 3.4.1.3.

$$c_{eff} = \begin{cases} \left(\frac{0.772}{\bar{\lambda}_l} - \frac{0.079}{\bar{\lambda}_l^2} \right) c \leq c & \text{for internal compression elements} \\ \left(\frac{1}{\bar{\lambda}_l} - \frac{0.188}{\bar{\lambda}_l^2} \right) c \leq c & \text{for outstand compression elements} \end{cases} \quad (3.2)$$

$$\bar{\lambda}_l = \frac{c/t}{28.4\varepsilon\sqrt{k_\sigma}} \quad (3.3)$$

3.4.1.2 Slenderness limits

The Eurocode Class 3 slenderness limit for outstand plate elements in compression was assessed based on the test and numerical data on angle and channel section stub columns and

angle section beams bent about the geometric axes in the ‘reverse L’ orientation, as shown in Figure 3.17, where the failure loads (or moments), normalised by the corresponding cross-section yield loads (or cross-section elastic moment resistances), are plotted against the $c/(t\epsilon)$ ratios of the outstand plate elements of the cross-sections (i.e. legs of angle sections or flanges of channel sections), together with the Eurocode Class 3 slenderness limit for outstand plate elements in compression $c/(t\epsilon)=14$; the results of the assessment revealed that the Eurocode Class 3 slenderness limit for outstand plate elements in compression can be accurately used for the classification of the legs of press-braked stainless steel angle sections and flanges of press-braked stainless steel channel sections. A similar graphic assessment was carried out on the Eurocode Class 3 slenderness limit for internal plate elements in compression $c/(t\epsilon)=37$, based on the test and numerical data on channel section beams in ‘n’-orientation minor-axis bending, as shown in Figure 3.18, indicating that the Eurocode Class 3 slenderness limit for internal plate elements subjected to compression is safe but conservative when applied to the webs of press-braked stainless steel channel sections.

Figure 3.17: Assessment of Class 3 slenderness limit for outstand elements in compression

Figure 3.18: Assessment of Class 3 slenderness limit for internal elements in compression

The test and numerical failure moments of angle section beams bent about the geometric axes in the ‘reverse L’ orientation are normalised by the corresponding plastic moment resistances and plotted against the $c/(t\epsilon)$ ratios of the angle legs in Figure 3.19, while the test and numerical failure moments of channel section beams in ‘n’-orientation minor-axis bending, normalised by the corresponding plastic moment capacities, are plotted against the $c/(t\epsilon)$ ratios of the channel webs in Figure 3.20. The Eurocode Class 2 slenderness limits for outstand plate elements in compression ($c/(t\epsilon)=10$) and internal plate elements in compression ($c/(t\epsilon)=35$), as also shown in Figures 3.19 and 3.20, were found to well capture the test and numerical data points, indicating a good level of accuracy.

Figure 3.19: Assessment of Class 2 slenderness limit for outstand elements in compression

Figure 3.20: Assessment of Class 2 slenderness limit for internal elements in compression

Assessment of both the Eurocode Class 2 slenderness limit and Class 3 slenderness limit for outstand plate elements under stress gradients (with tips in compression) were made, based on the test and numerical data on angle section beams bent in the ‘L’ orientation and channel section beams bent in the ‘u’ orientation. Figures 3.21 and 3.22 show the normalised failure

moments (by the plastic moment resistances and elastic moment resistances, respectively) plotted against the corresponding ratios of $\alpha c/(t\epsilon)$ and $c/(t\epsilon k_\sigma^{0.5})$, respectively, together with the Eurocode Class 2 and Class 3 slenderness limits for outstand plate elements under stress gradients (with tips in compression) $\alpha c/(t\epsilon)=10$ and $c/(t\epsilon k_\sigma^{0.5})=21$, where α is the ratio of the width of the compressive portion of the outstand plate element to the total width of the outstand plate element. The comparisons also indicated that these slenderness limits can be safely used for the classification of the outstand plate elements of press-braked stainless steel angle and channel sections.

Figure 3.21: Assessment of Class 2 slenderness limit for outstand elements under stress gradients (with tips in compression)

Figure 3.22: Assessment of Class 3 slenderness limit for outstand elements under stress gradients (with tips in compression)

3.4.1.3 Cross-section resistance predictions

The accuracy of the EC3 design cross-section resistances for press-braked stainless steel equal-leg angle and plain channel sections was assessed by comparing against the test (and numerical) failure loads and moments. The mean test and numerical to EC3 predicted failure load ratios $N_u/N_{u,EC3}$, as reported in Table 3.8(a), are respectively equal to 1.32 and 1.21 for non-slender and slender press-braked stainless steel equal-leg angle section (and plain channel section) stub columns, with the corresponding coefficients of variation (COVs) of 0.09 and 0.11, while the mean ratios of experimental and numerical to EC3 predicted failure moments $M_u/M_{u,EC3}$ are equal to 1.65 and 2.01 for non-slender and slender press-braked stainless steel equal-leg angle section (and plain channel section) beams, with the corresponding COVs of 0.27 and 0.07; both indicated the undue conservatism of the current EN 1993-1-4 (CEN, 2015) in the predictions of cross-section resistance of press-braked stainless steel open section structural members, mainly due to the neglect of the pronounced strain hardening of stainless steel. The conservatism and scatter of the EC3 design cross-section resistances is also indicated in Figure 3.23, where the test (and numerical) $N_u/N_{u,EC3}$ and $M_u/M_{u,EC3}$ ratios are plotted against the cross-

section slendernesses $\bar{\lambda}_p = \sqrt{\sigma_{0.2}/\sigma_{cr}}$, where σ_{cr} is the cross-section elastic critical local buckling stress subjected to the applied loading and obtained from the finite strip software CUFSM (Schafer and Ádány, 2006).

Table 3.8: Comparisons of test and numerical failure loads with predicted resistances

(a) EN 1993-1-4

Component type	Cross-section type	No. of test data	No. of FE data	$N_u/N_{u,EC3}$ or $M_u/M_{u,EC3}$	
				Mean	COV
Stub column	Non-slender	6	47	1.32	0.09
	Slender	2	76	1.21	0.11
	Sub-total	8	123	1.25	0.11
Beam	Non-slender	8	68	1.65	0.27
	Slender	2	35	2.01	0.07
	Sub-total	10	103	1.77	0.23

(b) SEI/ASCE 8-02 and AS/NZS 4673

Component type	Cross-section type	No. of test data	No. of FE data	$N_u/N_{u,ASCE}$ or $M_u/M_{u,ASCE}$ ($N_u/N_{u,AS/NZS}$ or $M_u/M_{u,AS/NZS}$)	
				Mean	COV
Stub column	Non-slender	6	74	1.48	0.17
	Slender	2	49	1.27	0.31
	Sub-total	8	123	1.40	0.23
Beam	Non-slender	8	74	2.07	0.06
	Slender	2	29	1.99	0.06
	Sub-total	10	103	2.05	0.06

(b) CSM

Component type	Cross-section type	No. of test data	No. of FE data	$N_u/N_{u,CSM}$ or $M_u/M_{u,CSM}$	
				Mean	COV
Stub column	Non-slender	6	65	1.12	0.04
	Slender	2	58	1.15	0.11
	Sub-total	8	123	1.13	0.08
Beam	Non-slender	8	96	1.07	0.07
	Slender	2	7	1.28	0.02
	Sub-total	10	103	1.08	0.08

Figure 3.23: Comparisons of press-braked stainless steel angle and channel section test (and numerical) results with EN 1993-1-4 and CSM resistance predictions

It is worth noting that for slender Class 4 angle sections bent about the geometric axes in the ‘reverse L’ orientation and slender Class 4 channel sections bent about the minor principal axes

in the ‘n’ orientation, where the neutral axes are closer to the extreme compressive fibres, although the compressive strains are less than yield strains, the tensile strains can be considerably greater than the yield strains (see Figure 3.24(a)), indicating that the tensile portions of slender angle and channel section beams subjected to bending can also benefit from material strain hardening, owing to which the ultimate moments of slender angle and channel sections can be even larger than the cross-section plastic moment resistances. For example, the ratios of the test failure moments to the cross-section plastic moment resistances are equal to 1.09 for the two slender (Class 4) angle section beam specimens A2-B1-rL and A2-B2-rL bent about the geometric axes in the ‘reverse L’ direction, as reported in Table 3.5. However, the current EN 1993-1-4 (CEN, 2015) neglects this favourable strain hardening effect associated with the tensile portions of slender stainless steel angle and channel section beams, and limits the corresponding cross-section bending moment resistances to the effective moment resistances, resulting in an overly high level of design conservatism.

Figure 3.24: EC3 and CSM design strain and stress distributions for a slender channel section in bending about the minor axis in the ‘n’ orientation

3.4.2 SEI/ASCE 8-02 (ASCE) and AS/NZS 4673 (AS/NZS)

The existing American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) use the same design rules for stainless steel structural members failing by local buckling. The design cross-section bending moment resistances of stainless steel equal-leg angle and plain channel sections with unstiffened plate elements were determined based on initiation of yielding, resulting in the elastic moment resistances for non-slender cross-sections without considering the beneficial plasticity and the effective moment resistances for slender cross-sections. Note that SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) employ the same effective width formulation for both the stiffened (i.e. internal) and unstiffened (i.e. outstand) plate elements, as defined by Equation (3.4), where c is taken as the flat element width excluding the corner radius. The design cross-section compression resistances of stainless steel plain channel section stub columns were specified as the yield loads for non-slender sections but determined from the effective width method for slender sections. With regards to concentrically loaded stainless steel angle section stub columns, both of the two standards specify that they should always be designed as eccentrically loaded structural members with the eccentricities to the cross-section minor principal axes equal to $L/1000$ (Liang et al., 2019a), based on the linear interaction formula defined by Equation (3.5), where $N_{u,ASCE}$ (or $N_{u,AS/NZS}$) is the design compression resistance, N_n is the nominal axial compression resistance and respectively equal to the yield load and effective compression resistance for non-slender and slender cross-sections, and M_{nz} is the cross-section bending moment resistance (i.e. elastic moment resistance for non-slender section and effective moment resistance for slender section).

$$c_{eff} = \left(\frac{1}{\lambda_l} - \frac{0.22}{\lambda_l^2} \right) c \leq c \quad (3.4)$$

$$\frac{N_{u,ASCE}}{N_n} + \frac{N_{u,ASCE}(L/1000)}{M_{nz}} = 1 \quad \text{or} \quad \frac{N_{u,AS/NZS}}{N_n} + \frac{N_{u,AS/NZS}(L/1000)}{M_{nz}} = 1 \quad (3.5)$$

The accuracy of SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) was assessed through comparing the test and numerical results against the design cross-section resistances. As reported in Table 3.8(b), the mean test and numerical to predicted compression

resistance ratios $N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$) are equal to 1.48 and 1.27, with the corresponding COVs of 0.17 and 0.31, for non-slender and slender press-braked stainless steel angle and channel section stub columns, respectively, while the mean ratios of $M_u/M_{u,ASCE}$ (or $M_u/M_{u,AS/NZS}$) are equal to 2.07 and 1.99, with the corresponding COVs of 0.06 and 0.06, for non-slender and slender press-braked stainless steel angle and channel section beams, respectively. SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) were found to yield even more conservative and scattered cross-section resistance predictions than EN 1993-1-4 (CEN, 2015). The inaccuracy of the ASCE and AS/NZS predicted cross-section resistances is also indicated in Figure 3.25, where the test (and numerical) to predicted resistance ratios are plotted against the cross-section slendernesses.

Figure 3.25: Comparisons of press-braked stainless steel angle and channel section test (and numerical) results with SEI/ASCE 8-02 (or AS/NZS 4673) and CSM resistance predictions

3.5 Continuous Strength Method

The continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) is an advanced deformation-based design approach, which relates the local buckling resistance of a structural member to its cross-section deformation capacity and uses an elastic, linear strain hardening material model to take into due account of strain hardening. The CSM cross-section deformation capacity, expressed in terms of the ratio of the cross-section limiting compressive strain to the yield strain $\varepsilon_{csm}/\varepsilon_y$, is determined by means of the base curve, as defined by Equation (3.6), while the elastic, linear strain hardening material model is characterised by four parameters (C_1 , C_2 , C_3 and C_4) and shown in Figure 3.26, where C_1 is adopted in Equation (3.6) to define a cut-off strain for the purpose of preventing over-predictions of the CSM design failure stresses, C_2 is used in Equation (3.7) to define the slope of the strain hardening stage E_{sh} of the CSM material model, and C_3 and C_4 are used to predict the material failure strain $\varepsilon_u = C_3(1 - \sigma_{0.2}/\sigma_u) + C_4$; for austenitic stainless steel grades, the

values of the four parameters C_1 , C_2 , C_3 and C_4 are equal to 0.1, 0.16, 1.0 and 0.0, respectively (Buchanan et al., 2016).

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \begin{cases} \frac{0.25}{\bar{\lambda}_p^{3.6}} \leq \min\left(15, \frac{C_1 \varepsilon_u}{\varepsilon_y}\right) & \text{for } \bar{\lambda}_p \leq 0.68 \\ \left(1 - \frac{0.222}{\bar{\lambda}_p^{1.05}}\right) \frac{1}{\bar{\lambda}_p^{1.05}} & \text{for } \bar{\lambda}_p > 0.68 \end{cases} \quad (3.6)$$

$$E_{sh} = \frac{\sigma_u - \sigma_{0.2}}{C_2 \varepsilon_u - \varepsilon_y} \quad (3.7)$$

Figure 3.26: CSM elastic, linear hardening material model

On the basis of the CSM material model, the CSM design ultimate stress at the limiting compressive strain can then be obtained from Equation (3.8). It is worth noting that for non-slender sections with the limiting compressive strain ratios greater than unity (i.e. the limiting compressive strains are greater than the yield strains), the calculated CSM design ultimate stresses exceed the material 0.2% proof stresses, allowing for the favourable effect of material strain hardening to be taken into account. The CSM design cross-section compression

resistance is then given as the CSM design ultimate stress multiplied by the gross section area, as shown in Equation (3.9).

$$\sigma_{csm} = \begin{cases} E\epsilon_{csm} & \text{for } \epsilon_{csm} < \epsilon_y \\ \sigma_{0.2} + E_{sh}(\epsilon_{csm} - \epsilon_y) & \text{for } \epsilon_{csm} \geq \epsilon_y \end{cases} \quad (3.8)$$

$$N_{u,csm} = A\sigma_{csm} \quad (3.9)$$

With regards to asymmetric angle sections in geometric-axis bending and singly symmetric channel sections in minor-axis bending, their design neutral axes are assumed to be located at the ENA for sections with $\bar{\lambda}_p > 0.6$ and at the mid-point between the ENA and PNA for sections with $\bar{\lambda}_p \leq 0.6$ in the CSM, following a comprehensive experimental and numerical verification (Zhao and Gardner, 2018). The CSM strain distribution is then obtained, based on an assumption that the strain distribution is linear-varying throughout the cross-section depth, as depicted in Figure 3.24(b), where $\epsilon_{csm,c}$ is the limiting compressive strain and calculated from the base curve given by Equation (3.6), y_c is distance from the outermost compressive fibre to the CSM design neutral axis, and $\epsilon_{csm,t}$ is the limiting tensile strain and determined from Equation (3.10). Then, the CSM design stress distribution can be determined, based on the CSM elastic, linear strain hardening material model, while the CSM design bending moment resistance was firstly integrated from the CSM design stresses over the cross-section depth and then transformed into a simplified formulation (Zhao and Gardner, 2018), as given by Equation (3.11), where $\epsilon_{csm,d}$ is the CSM design strain and taken as the maximum of $\epsilon_{csm,c}$ and $\epsilon_{csm,t}$, and α_{csm} is the CSM bending parameter, with the recommended values for angle sections in geometric-axis bending and channel sections in minor-axis bending shown in Table 3.9 (Zhao and Gardner, 2018). Note that the CSM is capable of considering the beneficial strain hardening effect associated with the tensile portions of slender angle and channel sections in bending, as shown in Figure 3.24(b), where the CSM design strain distribution and design stress distribution for a slender channel section in ‘n’-orientation minor-axis bending are shown.

$$\epsilon_{csm,t} = \frac{\epsilon_{csm,c}(B - y_c)}{y_c} \quad (3.10)$$

$$M_{u,csm} = \begin{cases} W_{el}\sigma_{0.2} \frac{\varepsilon_{csm,d}}{\varepsilon_y} & \text{for } \varepsilon_{csm,d} < \varepsilon_y \\ W_{pl}\sigma_{0.2} \left[1 + \frac{E_{sh}}{E} \frac{W_{el}}{W_{pl}} \left(\frac{\varepsilon_{csm,d}}{\varepsilon_y} - 1 \right) - \left(1 - \frac{W_{el}}{W_{pl}} \right) \right] / \left(\frac{\varepsilon_{csm,d}}{\varepsilon_y} \right)^{\alpha_{csm}} & \text{for } \varepsilon_{csm,d} \geq \varepsilon_y \end{cases} \quad (3.11)$$

Table 3.9: CSM bending parameter α_{csm}

Cross-section shape	Axis of bending	Aspect ratio	α_{csm}
Channel section	Minor axis	$H/B \leq 2$	1.5
		$H/B > 2$	1.0
Equal-leg angle section	Geometric axis	1.0	1.0

Numerical assessment of the CSM (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) was conducted through comparing the stub column and beam test (and numerical) results against the CSM design cross-section resistances, with the mean ratios of $N_u/N_{u,csm}$ and $M_u/M_{u,csm}$ reported in Table 3.8(c). The results of the assessment revealed that the CSM (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) results in accurate and consistent predictions of cross-section resistances for both non-slender and slender press-braked stainless steel equal-leg angle and plain channel section structural members, attributed to the rational exploitation of material strain hardening in the design. Graphic comparisons between the CSM and EC3 as well as ASCE (or AS/NZS) cross-section resistance predictions are also shown in Figures 3.23 and 3.25, respectively, indicating a substantially higher level of design accuracy and consistency of the CSM over the current design codes.

3.6 Concluding remarks

A systematic experimental and numerical study has been performed to investigate the local buckling behaviour and cross-section resistances of press-braked stainless steel equal-leg angle and plain channel sections. The experimental study included eight concentrically loaded stub column tests and ten laterally restrained beam tests (about the geometric axes for angle sections and minor principal axes for channel sections), together with material tensile coupon tests and

initial local geometric imperfection measurements, while the numerical study included a simulation study to replicate the test responses and a parametric study to generate further numerical data over a wide range of cross-section dimensions. The test and numerical results were used to assess the accuracy of the slenderness limits for classification of press-braked stainless steel outstand and internal plate elements, as set out in EN 1993-1-4 (CEN, 2015), and examine the efficiency of the codified local buckling design rules and continuous strength method in predicting cross-section resistances. The assessment results revealed that (i) the EC3 slenderness limits for outstand and internal plate elements in compression and stress gradients can be safely used for the classification of press-braked stainless steel equal-leg angle sections and plain channel sections, (ii) all the established design standards, including EN 1993-1-4 (CEN, 2015) , SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), result in overly conservative and scattered design cross-section resistances for press-braked stainless steel equal-leg angle and plain channel section stub columns and beams, mainly due to the neglect of the favourable strain hardening effect of stainless steel in the design, and (iii) the advanced continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) takes into due account of material strain hardening, and yields substantially improved design cross-section resistances over the established design standards.

CHAPTER 4

LOCAL BUCKLING BEHAVIOUR OF CHANNEL SECTIONS UNDER COMBINED LOADING ABOUT MAJOR AXIS

4.1 Introduction

Cold-formed steel channel section members are adopted in a wide range of structural applications, such as roof purlins, façade or greenhouse columns, lattice structures, chords in the framing structures and build-up columns. Stainless steel channel section members combine the advantage of stainless steel material and the simple geometric property of channel sections. At the cross-sectional level, the local buckling behaviour and resistances of press-braked stainless steel channel sections under pure compression and pure bending have been studied in Chapter 3, and Liang et al. (2019c, 2020) has also performed a systematic study on the structural performance and design of laser-welded stainless steel channel section stub columns under combined compression and bending, as reviewed in Chapter 2. However, the local buckling behaviour of press-braked stainless steel channel sections under combined

loading has been largely unexplored to date, which is therefore investigated and analysed in the current chapter and the following chapter.

In this chapter, a testing and numerical modelling programme is firstly conducted and reported. Specifically, the testing programme adopts two press-braked stainless steel channel sections, and for each cross-section, five identical stub column specimens are prepared and tested under eccentric compression with varying initial loading eccentricities. The numerical modelling programme includes a validation study, where finite element models are developed and validated against the test results, and a parametric study, where the validated finite element models are used to generate further numerical data over a wide range of cross-section dimensions and initial loading eccentricities. The obtained test and numerical data are used to assess the accuracy of the codified design rules, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), for press-braked stainless steel channel sections under combined compression and major-axis bending. Finally, a CSM-based design approach is proposed, and the accuracy, consistency and reliability of the new design proposals are assessed. The findings of this research have been reported by Zhang et al. (2021d).

4.2 Testing programme

4.2.1 Test specimens

A testing programme was firstly conducted to obtain a test data bank on press-braked stainless steel channel sections subject to combined compression and major-axis bending. Two channel sections C 80×50×6 and C 120×60×6, press-braked from grade EN 1.4420 austenitic stainless steel sheets, were adopted in the testing programme. For each channel section, five stub column specimens with the same nominal length were prepared and later tested under eccentric compression with different initial loading eccentricities; this resulted in a total of ten press-braked stainless steel channel section stub column specimens subject to major-axis combined loading. The specimen ID includes a number '1' or '2' ('1' signifying the C 80×50×6 specimen series and '2' signifying the C 120×60×6 specimen series) and a

letter from ‘A’ to ‘E’ (designating the specimens under eccentric compression with different initial loading eccentricities within each specimen series), e.g., 2D. The measured geometric dimensions of each press-braked stainless steel channel section stub column specimen are reported in Table 4.1.

Table 4.1: Measured geometric dimensions and initial local geometric imperfections of eccentrically loaded press-braked stainless steel channel section stub column specimens

Cross-section	Specimen ID	H (mm)	B (mm)	t (mm)	r (mm)	L (mm)	ω_{f1} (mm)	ω_{f2} (mm)	ω_w (mm)	ω_0 (mm)
C 80×50×6	1A	80.05	49.98	6.00	9.00	239.7	0.02	0.01	0.01	0.02
	1B	79.89	50.11	5.95	8.93	240.3	0.02	0.02	0.03	0.03
	1C	80.06	49.94	5.96	8.94	239.9	0.03	0.03	0.02	0.03
	1D	79.94	50.05	5.96	8.94	239.9	0.04	0.03	0.04	0.04
	1E	79.92	50.10	5.99	8.99	239.7	0.02	0.03	0.03	0.03
C 120×60×6	2A	120.16	59.88	5.98	8.97	360.0	0.03	0.02	0.02	0.03
	2B	119.94	60.12	5.99	8.99	359.9	0.02	0.02	0.02	0.02
	2C	120.17	59.94	5.97	8.96	360.2	0.03	0.04	0.03	0.04
	2D	119.89	59.98	5.99	8.99	359.6	0.04	0.03	0.03	0.04
	2E	119.92	60.08	6.00	9.00	359.7	0.02	0.02	0.03	0.03

4.2.2 Material tests

The detailed test setup and procedures for the material tensile flat and corner coupon tests are described in Chapter 3. The obtained (flat and corner) material stress–strain curves for the two adopted press-braked stainless steel channel sections are shown in Figures 3.3(c) and 3.3(d), respectively, while a summary of the key obtained material properties is given in Table 3.1.

4.2.3 Initial local geometric imperfection measurements

Initial local geometric imperfections can affect the structural behaviour of thin-walled steel structural members and lead to low resistances. Therefore, the initial local geometric imperfections of the ten press-braked stainless steel channel section stub column specimens were carefully measured prior to the combined loading tests. The test setup (Schafer and

Peköz, 1998b) is shown in Figure 4.1, where a stub column specimen is mounted onto the flat work bench of a CNC router and an LVDT, perpendicularly pointing to each of the constituent plates (i.e. two flanges and web), is moved along the longitudinal direction. It is also worth noting that the measurements were conducted at a distance of 30 mm away from the specimen ends, to avoid any welding effect induced from the process of welding of specimens with end plates. The magnitudes of the initial local geometric imperfections for each plate were determined as the deviations from the measured data to a linear regression line fitted to the data set, with the largest deviation reported in Table 4.1, where ω_{f1} , ω_{f2} and ω_w denote the largest deviations for the two flanges and the web, respectively, while the maximum of ω_{f1} , ω_{f2} and ω_w is taken as the initial local geometric imperfection magnitude of the specimen ω_0 .

Figure 4.1: Test setup for initial local geometric imperfection measurements

4.2.4 Combined loading tests

Eccentric compression tests were performed on the ten press-braked stainless steel channel section stub column specimens about the major axis to study their cross-section behaviour and resistances subject to combined compression and major-axis bending. For each of the two specimen series, a range of initial loading eccentricities were designed to obtain a wide range of ratios of compression load to major-axis bending moment. An INSTRON 5000 kN servo-

hydraulic testing machine was used for all eccentric compression tests, with the loading rate equal to 0.2 mm/min. Each end of the testing machine was equipped with a knife-edge device, comprising a pit plate with a semi-circular groove and a wedge plate with a knife-edge wedge, to offer pin-ended boundary conditions to the specimen ends about the major axis – see Figures 4.2(a) and 4.2(b), where a photograph and a schematic diagram of the eccentric compression test setup are shown, respectively. Prior to testing, each press-braked stainless steel channel section stub column specimen was welded to 15 mm thick steel plates at the ends, which helped to prevent the specimen ends from twisting and warping and also facilitate the application of pin-ended boundary conditions. Each specimen welded with end plates was then positioned between the top and bottom wedge plates, with their relative positions carefully adjusted to achieve the pre-specified initial loading eccentricity, and also ensure that the member longitudinal axis was perpendicular to the wedge plates and the cross-section major axis was parallel to the knife-edges. Upon completion of the position adjustment, the specimen was anchored by connecting the welded end plates with the wedge plates through bolts, before being placed between the top and bottom pit plates in the testing machine.

(a) Photograph

Figure 4.2: Eccentrically loaded stub column test setup

With regard to the instrumentation employed for the combined loading tests, as shown in Figures 4.2(a) and 4.2(b), one LVDT was horizontally pointed to the outer face of the flange at the specimen mid-height to measure the lateral deflections in the bending direction, while two strain gauges were respectively adhered to the outer faces of the two flanges at the specimen mid-height to record the maximum and minimum longitudinal strains. The readings from the LVDT and strain gauges were thereafter used to derive the actual initial loading eccentricity about the major axis e_0 , based on Equation (4.1) (Liang et al., 2019c), where N is the applied eccentric compression load, I_y is the second moment of area about the major principal axis, $\varepsilon_{max}-\varepsilon_{min}$ is the difference of the longitudinal strains measured by the two strain gauges, and Δ is the lateral deflection at the mid-height of the specimen recorded by the LVDT. Note that Equation (4.1) was derived based on the assumption that the structural behaviour was close to linear elastic, and it was thus suggested that no more than 15% of the expected failure load be used in the calculation of e_0 (Liang et al., 2019c).

$$e_0 = \frac{EI_y (\varepsilon_{max} - \varepsilon_{min})}{NH} - \Delta \quad (4.1)$$

The actual initial loading eccentricity e_0 , as calculated from Equation (4.1), together with the key results obtained from the combined loading tests, for each press-braked stainless steel

channel section stub column specimen are reported in Table 4.2, where $N_{u,test}$ is the failure load, Δ_u is the mid-height lateral deflection at the failure load and $M_u = N_{u,test}(e_0 + \Delta_u)$ is the failure moment at the specimen mid-height. The load–mid-height lateral deflection curves measured for the two series of eccentrically loaded press-braked stainless steel channel section stub column specimens are shown in Figures 4.3 and 4.4, respectively. It is evident that within each specimen series, the specimens with larger initial loading eccentricities have more ductile load–deformation responses, lower failure loads and larger deflections, due to the increased influence from major-axis bending. All the ten press-braked stainless steel channel section stub column specimens under major-axis combined loading failed by local buckling upon testing, featuring noticeable localised deformations at the compressive flanges and the compressive portions of the webs, as shown in Figures 4.5 and 4.6.

Table 4.2: Summary of major-axis combined loading test results

Cross-section	Specimen ID	e_0 (mm)	$N_{u,test}$ (kN)	Δ_u (mm)	M_u (kNm)
C 80×50×6	1A	8.3	327.8	4.3	4.1
	1B	22.1	254.4	4.9	6.9
	1C	30.4	212.0	4.9	7.5
	1D	40.7	188.1	4.9	8.6
	1E	52.7	164.0	5.2	9.5
C 120×60×6	2A	9.9	442.4	3.0	5.7
	2B	20.0	381.3	3.4	8.9
	2C	30.3	338.1	3.8	11.5
	2D	45.7	278.2	3.1	13.6
	2E	59.5	243.2	3.6	15.4

Figure 4.3: Load–mid-height lateral deflection curves for C 80×50×6 specimens

Figure 4.4: Load–mid-height lateral deflection curves for C 120×60×6 specimens

Figure 4.5: Experimental and numerical failure modes for specimen 1C

Figure 4.6: Experimental and numerical failure modes for specimen 2B

4.3 Numerical modelling programme

4.3.1 General

Following the testing programme, a supplementary numerical modelling programme was performed by using the nonlinear finite element software ABAQUS (ABAQUS, 2012). FE models of press-braked stainless steel channel section stub columns under combined loading about the major axis were firstly developed, as described in Section 4.3.2, while validation of the developed FE models against the experimental results was subsequently reported in Section 4.3.3. Finally, the validated FE models were used to perform parametric studies to generate further numerical data on press-braked stainless steel channel sections subject to combined compression and major-axis bending over a wide range of cross-section dimensions and initial loading eccentricities, as described in Section 4.3.4.

4.3.2 Development of FE models

For the development of FE models for press-braked stainless steel channel section stub columns under major-axis combined loading, the procedures and techniques of the selection of mesh size, modelling of material properties and incorporation of initial local geometric imperfections were the same as those described in Section 3.3.2. Regarding the modelling of boundary conditions, each end section of the channel section stub column FE model was coupled to an eccentric reference point, which was located eccentrically to the cross-section major axis, with the eccentricity equal to the corresponding measured value e_0 , and also offset longitudinally from the end section by 70 mm – the distance from the rotation centre of the knife-edge device to the end section of the specimen in the experiments (see Figures 4.2(a) and 4.2(b)). Proper boundary conditions were thereafter assigned to the reference points; one reference point was allowed for rotation about the major principal axis and translation in the longitudinal direction, while the other reference point was fully restrained except for rotation about the same major principal axis, to replicate the pin-ended boundary conditions offered by the knife-edges to the channel section stub column specimens about the major axis in the tests. Three local imperfection magnitudes, including the measured magnitude w_0 , and 1/10 and 1/100 of the wall thickness of the channel section, were adopted to factor the obtained

initial local geometric imperfection pattern, to assess the sensitivity of the developed press-braked stainless steel channel section stub column FE models to imperfection magnitudes. Upon incorporation of the initial local geometric imperfections into the FE models, nonlinear analyses were conducted using the modified Riks method (ABAQUS, 2012), to obtain the numerical failure loads, load–mid-height lateral deflection curves and failure modes.

4.3.3 Validation of FE models

The accuracy of the developed FE models was assessed through comparisons of the numerical results with those obtained from the tests. Table 4.4 reports the numerical to test failure load ratios $N_{u,FE}/N_{u,test}$ for the ten eccentrically loaded press-braked stainless steel channel section stub column specimens, with the results indicating that the most accurate test failure load predictions were obtained when the measured imperfection magnitudes were used, while the generalised imperfection magnitude of $t/100$ also resulted in a high degree of accuracy in the predictions of the test failure loads. Figures 4.5 and 4.6 show comparisons between the test and numerical failure modes for two typical press-braked stainless steel channel section stub column specimens 1C and 2B subject to major-axis combined loading, indicating excellent agreement. Moreover, the experimental load–mid-height lateral deflection curves were also accurately simulated by their numerical counterparts, as shown in Figures 4.7 and 4.8, where the experimental and numerical load–mid-height lateral deflection curves for the two aforementioned specimens 1C and 2B are compared. Overall, the developed FE models can accurately simulate the test responses of the press-braked stainless steel channel section stub column specimens subject to combined compression and major-axis bending and are thus deemed to be validated.

Table 4.3: Comparisons of numerical failure loads with test failure loads for varying initial local geometric imperfection magnitudes

Cross-section	Specimen ID	$N_{u,FE}/N_{u,test}$		
		ω_0	$t/10$	$t/100$
C 80×50×6	1A	0.969	0.948	0.969
	1B	0.960	0.936	0.967
	1C	1.016	0.984	1.015
	1D	0.983	0.957	0.983
	1E	0.969	0.944	0.969
C 120×60×6	2A	0.987	0.941	0.984
	2B	0.994	0.944	0.992
	2C	1.005	0.936	0.994
	2D	1.034	0.962	1.020
	2E	1.023	0.963	1.021
Mean		0.994	0.951	0.991
COV		0.025	0.016	0.021

Figure 4.7: Experimental and numerical load–mid-height lateral deflection curves for specimen 1C

Figure 4.8: Experimental and numerical load–mid-height lateral deflection curves for specimen 2B

4.3.4 Parametric studies

Having been validated in Section 4.3.3, the developed FE models were now used to perform parametric studies to generate further numerical data on eccentrically loaded press-braked stainless steel channel section stub columns over a wide range of cross-section dimensions and initial loading eccentricities. In the parametric studies, the adopted modelling procedures and techniques were the same as those previously described in Section 4.3.2, but with the measured flat and corner material stress–strain curves of the press-braked stainless steel channel section C 120×60×6 adopted in the material modelling and the initial local imperfection magnitudes fixed at $t/100$. With regard to the cross-section dimensions of the modelled channel sections, the outer web heights H were fixed at 150 mm, while the outer flange widths B varied from 50 mm to 150 mm, resulting in a wide variety of cross-section web-to-flange aspect ratios ranging between 1.0 and 3.0 being considered. Moreover, the wall thicknesses t were varied between 4 mm and 20 mm and the inner corner radii were set to be equal to 1.5 times the corresponding wall thicknesses. The numerically modelled channel sections covered all the four classes of cross-sections defined in EN 1993-1-4 (CEN, 2015)

and also non-slender and slender cross-sections specified in SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). For each modelled channel section, the corresponding model length was taken equal to three times the mean outer dimension of the web and flange. A broad variety of initial loading eccentricities varied from 1 mm to 600 mm were selected, which allowed different combinations of compression load and major-axis bending moment to be considered. In total, 390 numerical data were obtained through the parametric studies.

4.4 Assessment of current international design codes

4.4.1 General

In this section, the obtained test and numerical data were adopted to assess the accuracy and consistency of the existing codified design rules, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), for press-braked stainless steel channel sections subject to combined compression and major-axis bending. The unfactored design failure loads $N_{u,pred}$ were firstly determined from each design code and afterwards compared with the corresponding test and numerical failure loads N_u . The quantitative comparison results are reported in Table 4.4, where the mean $N_u/N_{u,pred}$ ratio and the corresponding COV for each codified design method are listed.

Table 4.4: Comparisons of test and numerical failure loads with predicted resistances from different design methods

No. of test data: 10 No. of FE data: 390	$N_u/N_{u,EC3}$	$N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$)	$N_u/N_{u,CSM}$
Mean	1.54	1.58	1.17
COV	0.08	0.13	0.08

4.4.2 EN 1993-1-4

The existing European code EN 1993-1-4 (CEN, 2015) provides no specific design rules for stainless steel cross-sections subject to combined loading, but directly adopts the corresponding design rules for carbon steel cross-sections subject to combined loading as

given in EN 1993-1-1 (CEN, 2014a). With regard to the design of channel sections subject to combined loading about the major axis, a linear design interaction formula is employed to predict their load-carrying resistances, as given by Equation (4.2),

$$\frac{N_{u,EC3}}{N_{EC3,Rd}} + \frac{M_{uy,EC3}}{M_{EC3,y,Rd}} \leq 1 \quad (4.2)$$

where $N_{u,EC3}$ is the design failure load, $M_{uy,EC3}$ is the design failure moment about the major axis and calculated as the product of the design failure load $N_{u,EC3}$ and the sum of the initial loading eccentricity e_0 and the mid-height lateral deflection at failure Δ_u , and $N_{EC3,Rd}$ and $M_{EC3,y,Rd}$ are the cross-section resistances in pure compression and pure bending about the major axis, respectively. For Class 1 and 2 channel sections, $N_{EC3,Rd}$ is equal to the cross-section yield load $N_{pl}=A\sigma_{0.2}$, and $M_{EC3,y,Rd}$ is taken as the plastic moment resistance $M_{pl,y}=W_{pl,y}\sigma_{0.2}$, where $W_{pl,y}$ is the plastic section modulus about the major axis. For Class 3 channel sections, $N_{EC3,Rd}$ remains equal to the cross-section yield load N_{pl} , while $M_{EC3,y,Rd}$ is now given by the elastic moment resistance $M_{el,y}=W_{el,y}\sigma_{0.2}$, where $W_{el,y}$ is the elastic section modulus about the major axis. For Class 4 channel sections, $N_{EC3,Rd}$ and $M_{EC3,y,Rd}$ are equal to the corresponding effective cross-section resistances $N_{eff}=A_{eff}\sigma_{0.2}$ and $M_{eff,y}=W_{eff,y}\sigma_{0.2}$, where $W_{eff,y}$ is the effective cross-section modulus, as determined based on the effective width method (CEN, 2015).

The accuracy of the EC3 design interaction formula was assessed through comparisons of the test and numerical failure loads with the design failure loads from Equation (4.2). As reported in Table 4.4, the mean test and numerical to EC3 predicted failure load ratio $N_u/N_{u,EC3}$ is equal to 1.54, with the corresponding COV of 0.08, revealing that the current EC3 design interaction formula yields overly conservative resistance predictions for press-braked stainless steel channel sections under combined compression and major-axis bending. The conservatism can also be seen from the graphical assessment results, as shown in Figure 4.9, where the test and numerical results, normalised by the respective yield loads and plastic moment capacities and arranged by cross-section classes, are plotted together with the EC3 average linear design interaction curves, and Figure 4.10, where the test and numerical failure

loads N_u are plotted against the EC3 design failure loads $N_{u,EC3}$. The conservative EC3 failure load predictions stems principally from the use of linear design interaction curves, which ignore the favourable spread of plasticity and stress redistribution within non-slender channel sections under combined loading, and from the inaccurate compression and bending end points of the linear design interaction curves, which are calculated without consideration of the pronounced strain hardening exhibited by stainless steel.

Figure 4.9: Eccentrically loaded stub column test and numerical results normalised by cross-section yield loads and plastic moment resistances

Figure 4.10: Comparison of test and numerical failure loads with predicted failure loads from EN 1993-1-4 and proposed approach

4.4.3 SEI/ASCE 8-02 and AS/NZS 4673

The American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) use the same design rules for cold-formed stainless steel structures. For a channel section under combined compression and major-axis bending moment, the design failure load $N_{u,ASCE}$ (or $N_{u,AS/NZS}$) should be calculated from a linear design interaction formula defined by Equation (4.3),

$$\frac{N_{u,ASCE}}{N_n} + \frac{M_{uy,ASCE}}{M_{ny}} \leq 1 \quad \text{or} \quad \frac{N_{u,AS/NZS}}{N_n} + \frac{M_{uy,AS/NZS}}{M_{ny}} \leq 1 \quad (4.3)$$

where $M_{uy,ASCE} = N_{u,ASCE}(e_0 + \Delta_u)$ (or $M_{uy,AS/NZS} = N_{u,AS/NZS}(e_0 + \Delta_u)$) is the design failure moment about the major axis, N_n is the nominal axial compression resistance and taken as the product of the 0.2% proof stress $\sigma_{0.2}$ and the full area A for non-slender sections or the effective area

A_{eff} for slender sections, and M_{ny} is the cross-section bending moment resistance about the major axis and should be determined based on initiation of yielding for channel sections with unstiffened (outstand flange) elements in compression, which therefore equals to the elastic moment resistance $M_{el,y}$ for non-slender channel sections but the effective moment resistance $M_{eff,y}$ for slender channel sections; it is worth noting that A_{eff} and $W_{eff,y}$ are now determined based on the ASCE (or AS/NZS) effective width method.

The ASCE (or AS/NZS) design failure loads for press-braked stainless steel channel sections subject to combined compression and bending moment about the major axis were assessed, based on the obtained test and numerical failure loads. Quantitative assessments of the ASCE (or AS/NZS) predicted failure loads are reported in Table 4.4, where the mean ratio of the test and numerical failure loads to the ASCE (or AS/NZS) predicted failure loads $N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$) is equal to 1.58, with the corresponding COV of 0.13, while the graphical assessments are shown in Figure 4.11, where the normalised test and numerical data points are compared with the average linear design interaction curves, and Figure 4.12, where the test and numerical failure loads N_u are plotted against the ASCE (or AS/NZS) design failure loads $N_{u,ASCE}$ (or $N_{u,AS/NZS}$). The results of the quantitative and graphical assessments generally indicated that the ASCE and AS/NZS linear design interaction curves lead to even more conservative and scattered failure load predictions for press-braked stainless steel channel sections subject to combined compression and bending moment about the major axis than their EC3 counterparts, owing mainly to the calculation of the bending end point based on initiation of yielding (i.e. plastic moment resistances are not allowed for).

Figure 4.11: Eccentrically loaded stub column test and numerical results normalised by cross-section yield loads and elastic moment resistances

Figure 4.12: Comparison of test and numerical failure loads with predicted failure loads from SEI/ASCE 8-02 (or AS/NZS 4673) and proposed approach

4.5 New design approach

4.5.1 General

As highlighted in Section 4, all the three considered codified design interaction curves (CEN, 2015; ASCE, 2002; AS/NZS, 2001) employ a linear shape, which cannot properly consider the beneficial effect of stress redistribution within channel sections, and are anchored at conservative compression and bending end points, which are calculated based on the 0.2% proof stress without exploitation of strain hardening of stainless steel, resulting in overly conservative and scattered predictions of failure load for press-braked stainless steel channel sections subject to combined compression and major-axis bending. In this section, to address the aforementioned shortcomings and limitations of the codified design interaction curves, a new design interaction curve was proposed by using a more efficient nonlinear shape and more accurate end points.

4.5.2 CSM for channel sections under isolated loading

The continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) is a deformation-based design method that allows to consider material strain hardening in the determination of cross-section compression and bending moment resistances. The method has been shown to result in a high level of accuracy and consistency in the prediction of stainless steel channel section resistances under isolated loading cases (i.e. pure compression (Afshan and Gardner, 2013; Zhao et al., 2017) and pure bending about the major axis (Zhao and Gardner, 2018)), which are ideal to be used as the new end points. The use of the CSM for the determination of cross-section resistances under isolated loading has been demonstrated in Section 3.5 in detail. The CSM cross-section compression resistance $N_{csm,Rd}$ is given as the product of the CSM design stress σ_{csm} and the full cross-section area A , as given by Equation (3.9), and the CSM cross-section bending moment resistance about the major axis $M_{csm,y,Rd}$ is given by Equation (4.4), where the CSM bending coefficient α_{csm} should now be taken as 2.0 for channel sections in bending about the major axis, and moreover, $W_{el}=W_{el,y}$ and $W_{pl}=W_{pl,y}$.

$$M_{csm,y,Rd} = \begin{cases} W_{el} \sigma_{0.2} \frac{\varepsilon_{csm,c}}{\varepsilon_y} & \text{for } \varepsilon_{csm,c} < \varepsilon_y \\ W_{pl} \sigma_{0.2} \left[1 + \frac{E_{sh}}{E} \frac{W_{el}}{W_{pl}} \left(\frac{\varepsilon_{csm,c}}{\varepsilon_y} - 1 \right) - \left(1 - \frac{W_{el}}{W_{pl}} \right) \right] / \left(\frac{\varepsilon_{csm,c}}{\varepsilon_y} \right)^{\alpha_{csm}} & \text{for } \varepsilon_{csm,c} \geq \varepsilon_y \end{cases} \quad (4.4)$$

4.5.3 CSM for channel sections under combined loading

The test and numerical results are normalised by the CSM cross-section compression and major-axis bending resistances and shown in Figure 4.13, where the normalised data points may now be seen to follow a much tighter trend in comparison to the rather scattered data points when normalised by the yield loads and plastic (or elastic) moment resistances in Figures 4.9 and 4.11. This indicates that the use of the CSM cross-section compression and bending moment resistances as the end points of the design interaction curves can substantially reduce the conservatism and scatter of the failure load predictions. The next step towards developing improved design approaches for press-braked stainless steel channel sections subject to major-axis combined loading now lies in the employment of efficient nonlinear interaction curves, anchored to these CSM end points.

Considering that the normalised test and numerical data points in Figure 4.13 exhibit a bi-linear distribution, it is proposed to adopt an interaction curve with a bi-linear shape anchored to the CSM end points for press-braked stainless steel channel sections with $\bar{\lambda}_p \leq 0.60$, as given by Equation (4.5), where $N_{u,csm}$ is the design failure load, $M_{uy,csm} = N_{u,csm}(e_0 + \Delta_u)$ is the design failure moment about the major axis, $M_{csm,y,Rd}$ is the CSM cross-section bending moment resistance about the major axis, while $M_{R,csm,y,Rd}$ is the reduced CSM cross-section bending moment resistance about the major axis due to the existence of the compression load $N_{u,csm}$, and a_w is the ratio of the web area to full area of the cross-section, with an upper limit of 0.5. It is worth noting that the proposed bi-linear interaction curve is similar to that derived based on full plasticity for I-sections subject to combined compression and bending about the major axis (CEN, 2014a). Then, for press-braked stainless steel channel sections with $\bar{\lambda}_p > 0.60$, a linear design interaction curve, defined by Equation (4.6), is recommended to be

used, to cater for the increasing elastic and linear structural behaviour. The proposed bi-linear and linear design interaction curves, displayed in Figure 4.13, were shown to follow closely the test and numerical data for press-braked stainless steel channel sections subject to combined loading about the major axis.

$$M_{uy, csm} \leq M_{R, csm, y, Rd} = M_{csm, y, Rd} \frac{1 - N_{u, csm} / N_{csm, Rd}}{1 - 0.5a_w} \leq M_{csm, y, Rd} \quad \text{for } \bar{\lambda}_p \leq 0.60 \quad (4.5)$$

$$\frac{N_{u, csm}}{N_{csm, Rd}} + \frac{M_{uy, csm}}{M_{csm, y, Rd}} \leq 1 \quad \text{for } \bar{\lambda}_p > 0.60 \quad (4.6)$$

The failure loads for press-braked stainless steel channel sections subject to combined compression and major-axis bending, as predicted from the new CSM-based design approach, were assessed against the test and numerical failure loads. The mean $N_u/N_{u, csm}$ ratio, as reported in Table 4.4, is equal to 1.17, with the corresponding COV of 0.08, indicating that the new CSM-based design approach leads to substantially more accurate failure load predictions for press-braked stainless steel channel sections subject to combined loading about the major axis than the current design codes. The improved design accuracy of the new CSM-based design approach can also be seen in Figures 4.10 and 4.12, where the test and numerical failure loads are plotted against the failure loads predicted from the new design proposal and existing design codes, and in Figure 4.14, where the normalised test data points are compared with the four considered interaction curves.

Figure 4.13: Eccentrically loaded stub column test and numerical results normalised by CSM cross-section compression and bending moment resistances

Figure 4.14: Comparison of test results with codified and proposed interaction curves

4.5.4 Reliability analysis

The reliability of the newly proposed CSM-based design approach was assessed through statistical analyses, according to the requirements of EN 1990 (CEN, 2002). In the present reliability analyses, the material over-strength ratio for austenitic stainless steel was taken as 1.3 with the COV of 0.06, while the COV of the geometric properties of stainless steel cross-sections was taken equal to 0.050 (Afshan et al., 2015). The key statistical parameters, calculated based on a total of 400 test and numerical data, are reported in Table 4.5, where $k_{d,n}$ is the design (ultimate limit state) fractile factor, b is the mean ratio of test and numerical resistances to design model resistances, V_δ is the COV of the test and numerical resistances relative to the resistance model, V_r is the combined COV incorporating both model and basic variable uncertainties, and γ_{M0} is the partial safety factor for stainless steel cross-section resistance. As reported in Table 4.5, the resulting partial safety factor for the new design proposal is equal to 0.95, which is less than the currently adopted value of 1.1 in EN 1993-1-4 (CEN, 2015), therefore demonstrating the reliability of the new design proposal for press-

braked stainless steel channel sections under combined compression and major-axis bending moment.

Table 4.5: Reliability analysis results calculated according to EN 1990

No. of test and FE data	$k_{d,n}$	b	V_δ	V_r	γ_{M0}
400	3.12	1.165	0.084	0.115	0.95

4.6 Concluding remarks

Laboratory testing and numerical modelling have been conducted to study the local buckling behaviour and resistances of press-braked stainless steel channel sections subject to combined compression and major-axis bending moment. The experimental programme included initial local geometric imperfection measurements and ten eccentrically loaded stub column tests on two press-braked stainless steel channel sections. The test results were used in a numerical modelling programme for the validation of FE models, which were then adopted to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions and initial loading eccentricities. The obtained test and numerical data were adopted to assess the accuracy of the current codified design rules, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), for press-braked stainless steel channel sections subject to combined compression and major-axis bending. The assessment results revealed that all the three examined codified design methods yield unduly conservative failure load predictions for press-braked stainless steel channel sections subject to major-axis combined loading, principally due to the adoption of conservative design interaction curves and the use of conservative compression and bending end points. A new design approach was finally proposed through the use of a bi-linear interaction curve, anchored to more accurate CSM end points, and shown to yield significantly improved failure load predictions for press-braked stainless steel channel sections under combined loading about the major axis. The reliability of the new design proposal was confirmed by means of statistical analyses according to the requirements given in EN 1990 (CEN, 2002).

CHAPTER 5

LOCAL BUCKLING BEHAVIOUR OF CHANNEL SECTIONS UNDER COMBINED LOADING ABOUT MINOR AXIS

5.1 Introduction

This chapter presents an experimental and numerical modelling programme to study the local buckling behaviour and resistances of press-braked channel sections under combined compression and bending moment about the minor axis. The experimental programme includes initial local geometric imperfection measurements and eccentric compression tests on ten press-braked stainless steel channel section stub column specimens. This is followed by a numerical modelling programme, where finite element models are initially developed and validated against the test results, and then used to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions and initial loading eccentricities. The obtained test and numerical data are used to assess the accuracy of the codified design provisions, as given in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE,

2002) and AS/NZS 4673 (AS/NZS, 2001), for press-braked stainless steel channel sections subject to combined compression and bending moment about the minor axis. An improved design approach is also proposed through extension of the deformation-based continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) to the case of press-braked stainless steel channel sections under combined compression and minor axis bending, and the accuracy and reliability of the new design approach are assessed, based on the test and numerical data. The findings of this research have been reported by Zhang et al. (2021c).

5.2 Testing programme

5.2.1 Test specimens

Given that no tests on press-braked stainless steel channel sections subject to combined compression and minor-axis bending have been reported to date, a testing programme was firstly conducted to obtain a test data bank. Two channel sections were adopted in the testing programme: C 80×50×6 and C 120×60×6, which were fabricated by press-braking from grade EN 1.4420 austenitic stainless steel plates. For each channel section, five stub column specimens with the same nominal length were prepared and later tested under eccentric compression with different initial loading eccentricities to the cross-section minor axis, resulting in a total of ten press-braked stainless steel channel section stub column specimens subject to minor-axis combined loading. The identifier of each specimen includes a number ‘1’ or ‘2’ (‘1’ signifying the C 80×50×6 specimen series and ‘2’ signifying the C 120×60×6 specimen series) and a letter from ‘A’ to ‘E’ (differentiating specimens with different initial loading eccentricities within each specimen series), e.g., 1A. Table 5.1 reports the measured cross-section dimensions and member length of each press-braked stainless steel channel section stub column specimen.

Table 5.1: Measured geometric dimensions and initial local geometric imperfections of eccentrically loaded press-braked stainless steel channel section stub column specimens

Cross-section	Specimen ID	H (mm)	B (mm)	t (mm)	r (mm)	L (mm)	ω_{f1} (mm)	ω_{f2} (mm)	ω_w (mm)	ω_0 (mm)
C 80×50×6	1A	80.08	49.97	5.96	8.94	239.7	0.01	0.02	0.01	0.02
	1B	80.11	49.89	5.95	8.93	240.3	0.03	0.03	0.02	0.03
	1C	79.94	50.06	5.96	8.94	239.9	0.02	0.02	0.03	0.03
	1D	80.06	49.93	6.00	9.00	239.9	0.03	0.04	0.04	0.04
	1E	79.92	50.10	5.99	8.99	239.7	0.03	0.03	0.03	0.03
C 120×60×6	2A	119.84	60.12	6.02	9.03	360.0	0.02	0.02	0.02	0.02
	2B	120.02	59.88	6.02	9.03	359.9	0.02	0.01	0.02	0.02
	2C	120.18	59.93	6.01	9.02	360.2	0.03	0.03	0.04	0.04
	2D	118.99	60.75	5.99	8.99	359.6	0.04	0.03	0.03	0.04
	2E	120.08	60.08	6.00	9.00	359.7	0.02	0.02	0.02	0.02

5.2.2 Material tests

The detailed test setup and procedures for the material tensile flat and corner coupon tests are described in Chapter 3. The obtained (flat and corner) material stress–strain curves for the two adopted press-braked stainless steel channel sections are shown in Figures 3.3(c) and 3.3(d), respectively, while a summary of the key obtained material properties is given in Table 3.1.

5.2.3 Initial local geometric imperfection measurements

Initial local geometric imperfections can affect the structural behaviour of thin-walled steel structural members and lead to low resistances. Therefore, the initial local geometric imperfections of the ten press-braked stainless steel channel section stub column specimens were carefully measured prior to the combined loading tests. The test setup and procedures were the same as those described in Section 4.2.3. The magnitudes of the initial local geometric imperfections for each plate were determined as the deviations from the measured data to a linear regression line fitted to the data set, with the largest deviation reported in Table 5.1, where ω_{f1} , ω_{f2} and ω_w denote the largest deviations for the two flanges and the web,

respectively, while the initial local geometric imperfection magnitude of the specimen ω_0 was taken as the maximum of ω_{f1} , ω_{f2} and ω_w .

5.2.4 Combined loading tests

Combined loading tests on the ten press-braked stainless steel channel section stub column specimens about the minor axis were conducted to study their cross-section behaviour and resistances under combined compression and minor-axis bending. The eccentric compression loads were applied to the stub column specimens with varying initial eccentricities to the cross-section elastic neutral axis, which led to a wide range of compression load-to-bending moment ratios being considered. It is also worth noting that when the eccentric compression loads were located between the cross-section elastic principal axes and flange tips, the resulting end moments induced the maximum compressive stresses at the flange tips; for the five C 80×50×6 stub column specimens, the eccentric compression loads were applied in this way. When the eccentric compression loads were located between the cross-section elastic principal axes and webs, the resulting end moments induced the maximum compressive stresses in the webs; for the five C 120×60×6 stub column specimens, the eccentric compression loads were applied in this way. The eccentrically loaded stub column tests were carried out using an INSTRON 5000 kN servo-hydraulic testing machine at a constant loading rate of 0.2 mm/min. Figures 5.1(a) and 5.1(b) show a photograph and a schematic diagram of the eccentric compression test setup, respectively. Each end of the testing machine was equipped with a knife-edge device, comprising a pit plate with a semi-circular groove and a wedge plate with a knife-edge wedge, to offer pin-ended boundary conditions to the specimen ends about the minor axis. Prior to testing, each press-braked stainless steel channel section stub column specimen was welded to 15 mm thick steel plates at the ends, which helped to prevent the specimen ends from twisting and warping and also facilitate the application of pin-ended boundary conditions. The channel section stub column specimen welded with end plates was then placed between the top and bottom wedge plates, and their relative position was carefully adjusted to achieve the pre-specified initial loading eccentricity, and also ensure that the member longitudinal axis was perpendicular to the wedge plates and the cross-section minor axis was parallel to the knife-edges. Upon

completion of the position adjustment, the specimen was anchored through bolting the end plates to the wedge plates.

(a) Photograph

(b) Schematic diagram

Figure 5.1: Eccentrically loaded stub column test setup

Figure 5.2: Locations of LVDT and strain gauges at specimen mid-height

With regard to the instrumentation employed for the combined loading tests, as shown in Figures 5.2, one LVDT, perpendicularly pointing to the centre line of the web at the specimen mid-height, was adopted to measure the lateral deflections about the minor principal axis, while two pairs of strain gauges (SG1 and SG2 as well as SG3 and SG4), adhered to the outer faces of the web and flange tips of the specimen at mid-height, were used to record the maximum and minimum longitudinal strains. The LVDT and strain gauge readings were thereafter used to derive the actual initial loading eccentricity from the cross-section elastic neutral axis e_{0e} , based on Equation (5.1) (Liang et al., 2019c), where I_z is the second moment of area about the minor principal axis, ϵ_{max} and ϵ_{min} are the average maximum compressive strain and the average minimum compressive strain (or the average maximum tensile strain, in the cases of large initial loading eccentricities) recorded by the two pairs of strain gauges, respectively, and Δ is the lateral deflection at the mid-height of the specimen recorded by the LVDT. Note that Equation (5.1) was derived based on the assumption that the structural behaviour was close to linear elastic, and it was thus suggested that no more than 15% of the expected failure load be used in the calculation of e_{0e} (Liang et al., 2019c).

$$e_{0e} = \frac{EI_z(\epsilon_{max} - \epsilon_{min})}{NB} - \Delta \quad (5.1)$$

The key test results for each eccentrically loaded press-braked stainless steel channel section stub column specimen are reported in Table 5.2, where e_{0e} is the actual initial loading eccentricity from the elastic neutral axis, $N_{u,test}$ is the failure load, Δ_u is the mid-height lateral

deflection at the failure load and $M_{ue}=N_{u,test}(e_{0e}+\Delta_u)$ is the failure moment at the specimen mid-height. Figures 5.3 and 5.4 show the experimental load–mid-height lateral deflection curves for the two series of press-braked stainless steel channel section stub column specimens under minor-axis combined loading, respectively. The deformed failure modes for the C 80×50×6 stub column specimens, where the eccentric compression loads were located between the cross-section elastic principal axes and flange tips, featured noticeable minor-axis bending in the ‘C’ orientation and localised deformations at the compressive flange tips, as shown in Figure 5.5. The deformed failure modes for the C 120×60×6 stub column specimens, where the eccentric compression loads were located between the cross-section elastic principal axes and webs, exhibited noticeable minor-axis bending in the ‘reverse C’ orientation and less visible localised deformations at the webs, as shown in Figure 5.6.

Table 5.2: Summary of minor-axis combined loading test results

Cross-section	Failure orientation	Specimen ID	e_{0e} (mm)	$N_{u,test}$ (kN)	Δ_u (mm)	M_{ue} (kNm)
C 80×50×6	C	1A	5.7	205.5	4.4	2.1
		1B	11.4	162.3	5.8	2.8
		1C	13.8	148.1	6.2	3.0
		1D	19.5	126.1	8.5	3.5
		1E	28.8	97.2	11.2	3.9
C 120×60×6	Reverse C	2A	12.5	332.4	5.4	5.9
		2B	16.6	280.5	6.5	6.5
		2C	21.9	240.8	6.5	6.8
		2D	25.6	215.6	7.0	7.0
		2E	38.4	155.2	8.4	7.3

Figure 5.3: Load–mid-height lateral deflection curves for C 80×50×6 specimens

Figure 5.4: Load–mid-height lateral deflection curves for C 120×60×6 specimens

Figure 5.5: Experimental and numerical failure modes for specimen 1B with failure in the ‘C’ orientation

Figure 5.6: Experimental and numerical failure modes for specimen 2A with failure in the ‘reverse C’ orientation

5.3 Numerical modelling programme

5.3.1 General

In conjunction with the testing programme, a numerical modelling programme was conducted by using the nonlinear finite element software ABAQUS (ABAQUS, 2012). Development of FE models for press-braked stainless steel channel section stub columns under combined loading about the minor axis was described in Section 5.3.2, while validation of the developed FE models against the experimental results was reported in Section 5.3.3. The validated FE models were then used to perform parametric studies to generate further numerical data on press-braked stainless steel channel section stub columns under minor-axis combined loading over a wide range of cross-section dimensions and initial loading eccentricities, as described in Section 5.3.4.

5.3.2 Development of FE models

For the development of FE models for press-braked stainless steel channel section stub columns under minor-axis combined loading, the procedures and techniques of the selection of mesh size, modelling of material properties and incorporation of initial local geometric imperfections were the same as those described in Section 3.3.2. Regarding the modelling of boundary conditions, the two end section of each channel section stub column FE model were firstly coupled to two reference points, which were located eccentrically to the cross-section minor axis, with the eccentricity taken as the corresponding measured value e_{0e} , and also offset longitudinally from the end sections by 70 mm, i.e. the distance from the rotation centre of the knife-edge device to the specimen end in the experiments – see Figures 5.1(a) and 5.1(b). Proper boundary conditions were thereafter assigned to the reference points; one reference point was allowed to translate in the longitudinal direction and rotate about the minor principal axis, while the other reference point was only allowed to rotate about the same minor principal axis, to replicate the pin-ended boundary conditions offered by the knife-edges to the channel section stub column specimens about the minor axis in the tests. Three local imperfection magnitudes, including the measured magnitude w_0 , and 1/10 and 1/100 of the wall thickness of the channel section, were adopted to factor the obtained initial

local geometric imperfection pattern, to assess the influence of the imperfection magnitudes on the structural behaviour of press-braked stainless steel channel sections under minor-axis combined loading.

5.3.3 Validation of FE models

Upon development of the eccentrically loaded press-braked stainless steel channel section stub column FE models, the modified Riks method (ABAQUS, 2012) was used to analyse the developed models and obtain the numerical failure loads, load–mid-height lateral deflection curves and failure modes. The accuracy of the developed FE models was assessed through comparisons of the numerical results against their experimental counterparts. Table 5.4 reports the numerical to test failure load ratios $N_{u,FE}/N_{u,test}$ for the ten eccentrically loaded press-braked stainless steel channel section stub column specimens, with the results generally revealing that (i) the test failure loads were well predicted for all the three employed local imperfection magnitudes, and (ii) the best agreement is achieved when the local imperfection magnitude is taken as ω_0 and $t/100$. The experimental and numerical failure modes for two typical specimens 1B (with failure in the ‘C’ orientation) and 2A (with failure in the ‘reverse C’ orientation) are compared in Figures 5.5 and 5.6, respectively, indicating excellent agreement. Moreover, Figures 5.7 and 5.8 show the test and numerical load–mid-height lateral deflection curves of the two aforementioned specimens 1B and 2A, where the test load–mid-height lateral deflection curves are well captured by their numerical counterparts. Overall, the developed FE models are capable of accurately simulating the test responses of the press-braked stainless steel channel section stub column specimens subject to combined compression and minor-axis bending, and thus considered to be validated.

Table 5.3: Comparisons of numerical failure loads with test failure loads for varying initial local geometric imperfection magnitudes

Cross-section	Failure orientation	Specimen ID	$N_{u,FE}/N_{u,test}$		
			ω_0	$t/10$	$t/100$
C 80×50×6	C	1A	0.954	0.951	0.956
		1B	0.962	0.955	0.964
		1C	0.973	0.967	0.973
		1D	0.968	0.961	0.968
		1E	1.014	0.999	1.013
C 120×60×6	Reverse C	2A	0.960	0.955	0.960
		2B	0.999	0.995	0.999
		2C	0.990	0.986	0.990
		2D	0.993	0.989	0.993
		2E	0.995	0.995	0.995
		Mean	0.981	0.975	0.981
		COV	0.020	0.020	0.020

Figure 5.7: Experimental and numerical load–mid-height lateral deflection curves for specimen 1B with failure in the ‘C’ orientation

Figure 5.8: Experimental and numerical load–mid-height lateral deflection curves for specimen 2A with failure in the ‘reverse C’ orientation

5.3.4 Parametric studies

The validated FE models were now used to perform parametric studies to generate further numerical data on press-braked stainless steel channel section stub columns subject to minor-axis combined loading over a wide range of cross-section dimensions and initial loading eccentricities. In the parametric studies, the measured flat and corner material stress–strain curves of the press-braked stainless steel channel section C 120×60×6 were adopted in the material modelling and the initial local imperfection magnitudes were taken as $t/100$. The cross-section dimensions were carefully selected to ensure that the numerically modelled channel sections covered Class 1–4 cross-sections defined in EN 1993-1-4 (CEN, 2015) and also fell into the category of non-slender and slender cross-sections specified in SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). Specifically, the outer web heights H were fixed at 150 mm, while the outer flange widths B varied from 50 mm to 150 mm, resulting in a wide variety of cross-section web-to-flange aspect ratios ranging between 1.0 and 3.0 being considered. Moreover, the wall thicknesses t were varied between 4 mm and 20 mm and the inner corner radii were set to be equal to 1.5 times the corresponding wall

thicknesses. The length of each FE model was equal to three times the mean outer dimension of the web and flange. Then, for each developed channel section stub column FE model, a wide range of initial loading eccentricities e_{0e} varied from 1 mm to 500 mm were selected and the initial loading eccentricities at both sides of the cross-section neutral axis were considered to study the behaviour and resistances of press-braked stainless steel channel sections subject to a broad range of combinations of compression and minor-axis bending and failing in both the ‘C’ and ‘reverse C’ orientations. In total, 780 numerical data on press-braked stainless steel channel section stub columns under minor-axis combined loading were generated from the parametric studies, with 390 for each failure orientation.

5.4 Assessment of current design codes and development of new design approach

5.4.1 General

In this section, the accuracy of the codified design rules for press-braked stainless steel channel sections subject to combined compression and minor-axis bending, as given in the European code EN 1993-1-4 (CEN, 2015), American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001), was assessed through comparisons of the test and numerical failure loads N_u with the unfactored design resistances $N_{u,pred}$. The quantitative comparison results for eccentrically loaded press-braked stainless steel channel section stub columns with failure in the ‘C’ and ‘reverse C’ orientations are reported in Table 5.4, including the mean $N_u/N_{u,pred}$ ratios and the corresponding COVs.

Table 5.4: Comparisons of test and numerical failure loads with predicted resistances from different design methods

(a) Failure in the 'C' orientation			
No. of test data: 5	$N_u/N_{u,EC3}$	$N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$)	$N_u/N_{u,csm}$
No. of FE data: 390			
Mean	1.79	1.94	1.08
COV	0.13	0.13	0.09

(b) Failure in the 'reverse C' orientation

No. of test data: 5	$N_u/N_{u,EC3}$	$N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$)	$N_u/N_{u,CSM}$
No. of FE data: 390			
Mean	2.33	2.68	1.14
COV	0.30	0.10	0.06

5.4.2 EN 1993-1-4

The European code for the design of stainless steel structures EN 1993-1-4 (CEN, 2015) employs the same design rules for the determination of cross-section resistances as those given in the carbon steel design code EN 1993-1-1 (CEN, 2014a). For channel sections under combined loading about the minor axis, a linear design interaction formula is used, as given by Equation (5.2), where $N_{u,EC3}$ is the design failure load, $M_{uz,EC3}=N_{u,EC3}(e_0+A_u)$ is the design failure moment about the minor axis, $N_{EC3,Rd}$ is the cross-section compression resistance, and $M_{EC3,z,Rd}$ is the cross-section bending moment resistance about the minor axis. The values of e_0 , $N_{EC3,Rd}$ and $M_{EC3,z,Rd}$ depend on the cross-section class. Specifically, for Class 1 and 2 channel sections, where the design neutral axis is assumed to be located at the cross-section plastic neutral axis, e_0 is taken as the distance from the initial loading point to the PNA, $N_{EC3,Rd}$ is equal to the cross-section yield load $N_{pl}=A\sigma_{0.2}$, and $M_{EC3,z,Rd}$ is taken as the plastic moment resistance $M_{pl,z}=W_{pl,z}\sigma_{0.2}$, where $W_{pl,z}$ is the plastic section modulus about the minor axis. For Class 3 channel sections, where the elastic neutral axis is assumed to be the design neutral axis at failure, e_0 is taken as the distance from the initial loading point to the ENA, $N_{EC3,Rd}$ remains equal to the cross-section yield load N_{pl} , while $M_{EC3,z,Rd}$ is now given by the elastic moment resistance $M_{el,z}=W_{el,z}\sigma_{0.2}$, where $W_{el,z}$ is the elastic section modulus about the minor axis. For Class 4 channel sections, e_0 is equal to the distance from the initial loading point to the effective neutral axis (EFNA), and the effective cross-section area A_{eff} and modulus $W_{eff,z}$ are employed in place of the gross cross-section properties for the calculation of $N_{EC3,Rd}$ and $M_{EC3,z,Rd}$.

$$\frac{N_{u,EC3}}{N_{EC3,Rd}} + \frac{M_{uz,EC3}}{M_{EC3,z,Rd}} \leq 1 \quad (5.2)$$

The accuracy of the EC3 design interaction formula was assessed through comparisons of the test and numerical failure loads with the design failure loads from Equation (5.2). As reported in Table 5.4, the mean test and numerical to EC3 predicted failure load ratios $N_u/N_{u,EC3}$ are equal to 1.79 and 2.33, with the corresponding COVs of 0.13 and 0.30, revealing that the European code EN 1993-1-4 (CEN, 2015) yields rather conservative and scattered resistance predictions for press-braked stainless steel channel sections under combined compression and minor-axis bending moment in both of the ‘C’ and ‘reverse C’ orientations. The above discussions are also supported by the graphical assessment results shown in Figure 5.9, where the test and numerical results are normalised by the respective yield loads and plastic moment resistances and plotted against the average linear design interaction curves obtained from EN 1993-1-4 (CEN, 2015). The conservative and scattered resistance predictions are attributed to the shortcomings of the EC3 design interaction formula, including (i) the linear design interaction curve neglects the beneficial stress redistribution that takes place within non-slender channel sections under combined loading, and (ii) the compression and bending end points of the design interaction curve are limited to the cross-section yield load and plastic (or elastic and effective) moment resistances, without accounting for the pronounced strain hardening exhibited by stainless steel.

(a) Failure in the ‘C’ orientation

(b) Failure in the ‘reverse C’ orientation

Figure 5.9: Comparisons of test and numerical data with EC3 design interaction curves

5.4.3 SEI/ASCE 8-02 and AS/NZS 4673

The American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) use the same design provisions for cold-formed stainless steel cross-sections subject to combined loading. Specifically, a linear design interaction formula is used for the design of cold-formed stainless steel channel sections subject to combined loading about the minor axis, as defined by Equation (5.3),

$$\frac{N_{u,ASCE}}{N_n} + \frac{M_{uz,ASCE}}{M_{nz}} \leq 1 \quad \text{or} \quad \frac{N_{u,AS/NZS}}{N_n} + \frac{M_{uz,AS/NZS}}{M_{nz}} \leq 1 \quad (5.3)$$

where $N_{u,ASCE}$ (or $N_{u,AS/NZS}$) is the design failure load, $M_{uz,ASCE}=N_{u,ASCE}(e_0+\Delta_u)$ (or $M_{uz,AS/NZS}=N_{u,AS/NZS}(e_0+\Delta_u)$) is the design failure moment about the minor axis, N_n is the nominal axial compression resistance and taken as the product of the 0.2% proof stress $\sigma_{0.2}$

and the full area A for non-slender sections or the effective area A_{eff} for slender sections, and M_{nz} is the cross-section bending moment resistance about the minor axis and should be determined based on initiation of yielding for channel sections with unstiffened (outstand flange) elements in compression, resulting in the elastic and effective moment resistances $M_{el,z}$ and $M_{eff,z}$ being adopted for non-slender and slender channel sections, respectively. It is worth noting that (i) the effective section properties (A_{eff} and $W_{eff,z}$) are now determined by the AS/NZS (or ASCE) effective width equations (ASCE, 2002; AS/NZS, 2001), which are different to their EC3 counterparts (CEN, 2006, 2015), and (ii) e_0 is taken as e_{0e} for non-slender channel sections but e_{0eff} for slender channel sections (which is also calculated using the AS/NZS (or ASCE) effective width equations).

The accuracy of the ASCE (or AS/NZS) failure load predictions for press-braked stainless steel channel sections subject to minor-axis combined loading were assessed through comparisons with the test and numerical failure loads. The mean ratios of $N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$), as reported in Table 5.5, are equal to 1.94 and 2.68, with the corresponding COVs of 0.13 and 0.10 for press-braked stainless steel channel sections under combined compression and minor-axis bending moment in the ‘C’ and ‘reverse C’ orientations, respectively. The graphical assessment results are shown in Figure 5.10, where the normalised test and numerical data are compared with the AS/NZS (or ASCE) average linear interaction curves. The American specification SEI/ACSE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) were generally found to result in even more conservative failure load predictions for press-braked stainless steel channel sections under combined compression and bending moment about the minor axis than the European code EN 1993-1-4 (CEN, 2015), owing principally to the use of more conservative bending end point, which is limited to the cross-section elastic moment resistance for non-slender channel sections.

(a) Failure in the 'C' orientation

(b) Failure in the 'reverse C' orientation

Figure 5.10: Comparisons of test and numerical data with ASCE (or AS/NZS) design interaction curves

5.4.4 New design approach

On the basis of the quantitative and graphical assessment results, it was found that the codified design interaction curves for press-braked stainless steel channel sections under the combined compression and bending moment about the minor axis, as given in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), suffer from having inaccurate shapes and end points; the shapes do not adequately take into account the favourable stress redistribution within channel sections subjected to combined loading, while the compression and bending end points of the design interaction curves are calculated based on the 0.2% proof stress without considering the favourable material strain hardening of stainless steel. Therefore, improved design approaches may be sought through the use of more accurate end points and then the development of more efficient nonlinear interaction curves, anchored to these new end points.

The continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) is a deformation-based design method that allows account to be taken of strain hardening in the determination of cross-section compression and bending moment resistances. Comparisons of test and numerical results on press-braked stainless steel channel section stub columns and beams with the CSM resistance predictions generally indicated a high level of design accuracy and consistency, as discussed in Chapter 3. Therefore, the CSM cross-section compression and bending moment resistances are ideal end points for the improved design interaction curves. The use of the CSM for the determination of cross-section resistances under isolated loading has been demonstrated in Section 3.5 in detail. The CSM cross-section compression resistance $N_{csm,Rd}$ is given by Equation (3.9), while the CSM cross-section bending moment resistance $M_{csm,z,Rd}$ is given by Equation (3.11), where $W_{el}=W_{el,z}$ and $W_{pl}=W_{pl,z}$.

The test and numerical failure loads and moments have been normalised by the CSM cross-section compression and bending moment resistances, and plotted in Figures 5.11(a) and 5.11(b). The normalised data points may be seen to follow a much tighter trend in comparison to those normalised by the cross-section yield load and plastic (or elastic) bending moment resistance in Figures 5.9 and 5.10. This demonstrated that the CSM cross-

section compression and bending resistances are ideal to be used as the end points. Considering the general distribution of the normalised test and numerical data points in Figures 5.11(a) and 5.11(b), it is proposed to adopt a new nonlinear interaction curve, as defined by Equation (5.4), but with the CSM cross-section resistances as the end points, for press-braked stainless steel channel sections with $\bar{\lambda}_p \leq 0.60$, in which $N_{u,csm}$ is the design failure load and $M_{uz,csm} = N_{u,csm}(e_0 + \Delta_u)$ is the design failure moment about the minor axis; note that e_0 is now taken equal to the distance from the initial loading point to the mid-point between ENA and PNA. For press-braked stainless steel channel sections with $\bar{\lambda}_p > 0.60$, the corresponding cross-section interaction behaviour becomes increasingly linear, and a linear design interaction curve is thus adopted, as given by Equation (5.5), where both the design applied bending moment $M_{uz,csm} = N_{u,csm}(e_0 + \Delta_u)$ and the CSM cross-section bending moment resistance $M_{csm,z,Rd}$ are calculated based on the design neutral axis located at the ENA. The proposed interaction curves, as plotted in Figures 5.11(a) and 5.11(b), were shown to well follow the distribution of the data points for press-braked stainless steel channel sections subject to combined compression and minor-axis bending in both the ‘C’ and ‘reverse C’ orientations.

$$\frac{M_{uz,csm}}{M_{csm,z,Rd}} = 1 - \left(\frac{N_{u,csm}}{N_{csm,Rd}} \right)^2 \quad \text{for } \bar{\lambda}_p \leq 0.60 \quad (5.4)$$

$$\frac{N_{u,csm}}{N_{csm,Rd}} + \frac{M_{uz,csm}}{M_{csm,z,Rd}} \leq 1 \quad \text{for } \bar{\lambda}_p > 0.60 \quad (5.5)$$

(a) Failure in the 'C' orientation

(b) Failure in the 'reverse C' orientation

Figure 5.11: Comparisons of test and numerical data with proposed design interaction curves

Quantitative and graphical assessments of the new design proposal for press-braked stainless steel channel sections subject to minor-axis combined loading were conducted. As reported in Table 5.4, the mean $N_u/N_{u,csm}$ ratios are equal to 1.08 and 1.14, with the corresponding COVs of 0.09 and 0.06, for the cases of minor-axis combined loading in the ‘C’ and ‘reverse C’ orientations, respectively, indicating that the proposed interaction curves lead to substantially improved design accuracy and consistency over their codified counterparts. This can also be evidenced from the graphical assessment results, as shown in Figures 5.12 and 5.13, where the test and numerical failure loads are plotted against the predicted failure loads from the proposed and codified interaction curves.

(a) Failure in the ‘C’ orientation

Figure 5.12: Comparisons of test and numerical failure loads with predicted resistances from EC3 and proposed design interaction curves

(b) Failure in the 'reverse C' orientation

Figure 5.13: Comparisons of test and numerical failure loads with predicted resistances from ASCE (or AS/NZS) and proposed design interaction curves

The reliability of the new design proposal was assessed through statistical analyses, according to the requirements of EN 1990 (CEN, 2002), as detailed in Section 4.5.4. The key statistical parameters, calculated based on a total of 790 test and numerical data, are reported in Table 5.5. The resulting (required) partial safety factor for the new design proposal is equal to 0.95, which is less than the currently adopted value of 1.1 in EN 1993-1-4 (CEN, 2015), therefore demonstrating the fact that the new design proposal for press-braked stainless steel channel sections under combined compression and minor-axis bending moment satisfies the reliability requirements of EN 1990 (CEN, 2002).

Table 5.5: Reliability analysis results calculated according to EN 1990

No. of test and FE data	$k_{d,n}$	b	V_{δ}	V_r	γ_{M0}
790	3.10	1.152	0.083	0.114	0.95

5.5 Concluding remarks

A comprehensive experimental and numerical modelling programme has been conducted to investigate the local buckling behaviour and resistances of press-braked stainless steel channel sections under combined compression and minor-axis bending moment. The experimental programme included initial local geometric imperfection measurements and ten eccentrically loaded press-braked stainless steel channel section stub column tests about the minor axis. The test setup, procedures and results, including the failure loads, load–mid-height lateral deflection curves and local buckling failure modes in both ‘C’ and ‘reverse C’ orientations, were fully reported. Following the experimental programme, a numerical modelling programme was conducted; FE models were firstly developed and validated against the test results, and then adopted to perform parametric studies to generate further numerical data on press-braked stainless steel channel section columns over a wide range of cross-section dimensions and initial loading eccentricities. The obtained test and numerical results were then employed to assess the accuracy of the existing design provisions for press-braked stainless steel channel sections under combined compression and minor-axis bending moment, as given in the European code EN 1993-1-4 (CEN, 2015), American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001), revealing rather conservative and scattered resistance predictions; this can be attributed to the adoption of inaccurate design interaction curves and the employment of conservative cross-section compression and bending resistances as the end points that are determined without accounting for strain hardening. Finally, an improved design approach was proposed through the use of a nonlinear interaction curve, anchored to more accurate end points determined from the continuous strength method (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018), which rationally exploits strain hardening in the determination of cross-section compression and bending resistances. The new design approach was shown to result in accurate, consistent and reliable failure load predictions for press-braked stainless steel channel sections subject to combined loading about the minor axis.

CHAPTER 6

FLEXURAL-TORSIONAL BUCKLING BEHAVIOUR OF FIXED-ENDED ANGLE SECTION COLUMNS

6.1 Introduction

Angle section compression and tension members are commonly used in frame structures and transmission towers as lateral bracing chords due to their simple cross-section shapes (enabling easy fabrication and assembly). However, the buckling behaviour of angle section members is generally complicated, mainly owing to the low torsion constants associated with the open and non-doubly symmetric cross-section shapes; angle section members are therefore vulnerable to torsion-related buckling, and much more tedious design procedures are required, compared to those for doubly symmetric tubular section members. Although there have been extensive and comprehensive studies on cold-formed carbon steel angle section columns, research into their stainless steel counterparts remains scarce, as highlighted in Chapter 2.

The local buckling behaviour and resistances of press-braked stainless steel angle sections have been studied in Chapter 3 through stub column tests and in-plane bending tests, while this chapter reports an experimental and numerical investigation of the flexural-torsional buckling behaviour of fixed-ended press-braked stainless steel equal-leg angle section columns. The experimental programme includes material tensile flat and corner coupon tests, initial global and torsional geometric imperfection measurements and sixteen fixed-ended column tests, while the finite element modelling programme includes a validation study and a parametric study to generate further numerical data over a wide range of cross-section dimensions and member lengths. The obtained test and numerical results are adopted to assess the accuracy of the established design provisions for fixed-ended press-braked stainless steel equal-leg angle section columns prone to flexural-torsional buckling, as given in the European code EN 1993-1-4 (CEN, 2015), American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001). Extension of the direct strength method to carbon steel equal-leg angle section columns has been recently made (Dinis and Camotim, 2015; Landesmann et al., 2017), and the applicability of the proposal to fixed-ended press-braked stainless steel equal-leg angle section columns is also assessed. A new method for the design of fixed-ended press-braked stainless steel angle section columns prone to flexural-torsional buckling is finally derived, followed by the assessment of the design accuracy and reliability. The findings of this research have been reported by Zhang et al. (2019a).

6.2 Testing programme

6.2.1 General

A testing programme was firstly performed to study the flexural-torsional buckling behaviour and resistances of fixed-ended press-braked stainless steel equal-leg angle section columns. Two equal-leg angle sections – A 50×50×6 and A 100×100×6, press-braked from EN 1.4420 austenitic stainless steel sheets, were employed in the present experimental study. The overall testing programme included tensile coupon tests to obtain the material responses of the two

examined angle sections, initial global and torsional geometric imperfection measurements to determine the initial global and torsional geometric imperfections of the specimens, and sixteen fixed-ended column tests to study the flexural-torsional buckling behaviour and resistances of press-braked stainless steel equal-leg angle section columns.

6.2.2 Material tests

The detailed test setup and procedures for the material tensile flat and corner coupon tests are described in Chapter 3. The obtained (flat and corner) material stress–strain curves for the two adopted press-braked stainless steel angle sections are shown in Figures 3.3(a) and 3.3(b), respectively, while a summary of the key obtained material properties is given in Table 3.1.

6.2.3 Initial global and torsional geometric imperfection measurements

Initial geometric imperfections can greatly affect the structural performance of thin-walled steel section members, leading to low load-carrying and deformation capacities and steep post-ultimate responses. Initial global and torsional geometric imperfections for each angle section specimen were thus carefully measured prior to testing. The test setup for initial global and torsional geometric imperfection measurements is shown in Figure 6.1, where two LVDTs (denoted by LVDT-1 and LVDT-2 or LVDT-3 and LVDT-4) were located at a distance of 10 mm from the two edges of each angle leg, to determine the deviations along two longitudinal lines. Figure 6.2 shows the configurations of the outer faces of a perfect ‘reference’ angle section located at the starting point of imperfection measurements and an imperfect ‘deformed’ angle section along the direction of imperfection measurements, deviated from the reference angle section. On the basis of the readings from the four LVDTs, the initial torsional geometric imperfection β , initial global geometric imperfections about the cross-section geometric axes (d_H and d_V) and initial global geometric imperfections about the cross-section major and minor principal axes (d_M and d_m) can be derived (Landesmann et al., 2017), as given by Equations (6.1)–(6.5), respectively, where l_1 , l_2 , l_3 and l_4 are the readings of LVDT-1, LVDT-2, LVDT-3 and LVDT-4, respectively, and y_1 , y_2 , x_3 and x_4 are the

distances from the locations of the four LVDTs to the corner of the angle section, respectively (see Figure 6.2). Figure 6.3 shows the longitudinal distributions of the derived initial global geometric imperfections about the major and minor principal axes and initial torsional geometric imperfection for a typical press-braked stainless steel angle section column specimen, while the measured maximum amplitudes of the three types of initial geometric imperfections (denoted as ω_{dM} , ω_{dm} and ω_{β} , respectively) for each tested angle section column specimen are reported in Table 6.1.

$$\beta = \tan^{-1}\left(\frac{l_1 - l_2}{y_2 - y_1}\right) \quad \text{or} \quad \beta = \tan^{-1}\left(\frac{l_4 - l_3}{x_4 - x_3}\right) \quad (6.1)$$

$$d_H = (l_1 - l_4 \tan \beta + y_1 \tan \beta + x_4 \tan^2 \beta) / (1 + \tan^2 \beta) \quad (6.2)$$

$$d_V = (l_1 \tan \beta + l_4 + y_1 \tan^2 \beta - x_4 \tan \beta) / (1 + \tan^2 \beta) \quad (6.3)$$

$$d_M = (d_H - d_V) / \sqrt{2} \quad (6.4)$$

$$d_m = (d_H + d_V) / \sqrt{2} \quad (6.5)$$

Figure 6.1: Test setup for initial global and torsional geometric imperfection measurements

Figure 6.2: Configuration of the outer faces of 'undeformed' and 'deformed' cross-sections

Figure 6.3: Typical measured initial global and torsional geometric imperfection distributions for specimen A100-1100

Table 6.1: Measured geometric dimensions and initial geometric imperfections of angle section column specimens

Cross-section	Specimen ID	L (mm)	B (mm)	t (mm)	r (mm)	ω_{dM} (mm)	ω_{dm} (mm)	ω_{β} (deg)
A 50×50×6	A50-150	149.0	49.98	5.92	8.88	0.05	0.02	0.08
	A50-150-R	149.9	49.87	5.90	8.85	0.05	0.01	0.07
	A50-300	299.6	49.98	5.90	8.85	0.04	0.05	0.08
	A50-360	359.6	49.78	5.93	8.90	0.04	0.09	0.10
	A50-420	419.8	50.06	5.94	8.91	0.03	0.11	0.10
	A50-480	478.9	50.03	5.88	8.82	0.05	0.19	0.09
	A50-540	539.8	49.91	5.97	8.96	0.10	0.19	0.11
	A50-600	599.5	49.94	5.90	8.85	0.08	0.23	0.10
A 100×100×6	A100-300	298.4	99.90	5.80	8.70	0.02	0.01	0.03
	A100-300-R	298.2	100.00	5.97	8.96	0.01	0.07	0.07
	A100-500	497.3	99.95	5.88	8.82	0.06	0.08	0.10
	A100-700	700.2	99.80	5.89	8.84	0.03	0.11	0.11
	A100-900	899.3	100.12	5.89	8.84	0.03	0.05	0.02
	A100-1100	1099.0	99.95	5.84	8.76	0.08	0.07	0.05
	A100-1300	1298.7	100.02	5.89	8.84	0.06	0.04	0.09
	A100-1500	1499.3	100.02	5.91	8.87	0.17	0.03	0.09

Note: ‘R’ indicates a repeated test.

6.2.4 Fixed-ended column tests

Sixteen fixed-ended column tests were carried out to study the flexural-torsional buckling behaviour and resistances of press-braked stainless steel equal-leg angle section columns. For each of the two considered angle sections, seven columns with varying member lengths were employed in the testing, with an additional repeated test conducted on the shortest column specimen. The column lengths were carefully selected, based on a prior elastic critical buckling analysis (Landesmann et al., 2017), and fell within the ‘plateau’ of the elastic critical buckling stress–column length curve, so that flexural-torsional buckling mode is the first critical buckling mode. The labelling system of the specimens starts with a letter ‘A’ (indicating angle section), followed by the nominal leg width of the angle, and ends with the nominal member length, e.g., A50-150. Table 6.1 reports the measured cross-section dimensions, member length and maximum initial global and torsional geometric imperfections for each column specimen, where ω_{β} is the maximum initial torsional

imperfection, and ω_{dM} and ω_{dm} are the maximum initial global imperfections about the major and minor principal axes, respectively.

The fixed-ended angle section column tests were carried out using an INSTRON 2000 kN hydraulic testing machine at a constant loading rate of 0.2 mm/min. Figure 6.4 shows the fixed-ended column test setup. Specifically, an anchor device is placed at the specimen ends to prevent end section translations and rotations about the two principal axes as well as torsional rotation, to achieve fixed-ended boundary conditions in the testing. Two strain gauges are adhered to the column specimen at mid-height to record the longitudinal (axial) strains during testing. A pair of LVDTs is positioned vertically at one end of the specimen to measure the member end shortening, while another two pairs of LVDTs are located horizontally at a distance of 10 mm from the two edges of the angle legs at mid-height, to track the translations at these positions. The translations measured from the two pairs of horizontally-placed LVDTs were then used to determine the lateral deflections along the two principal axes and torsional rotation at the mid-height of each angle section column specimen, following the same procedure and equations used for the calculation of initial global and torsional geometric imperfections in Section 6.2.3.

Figure 6.4: Fixed-ended column test setup

The key obtained test results for the fixed-ended press-braked stainless steel equal-leg angle section columns are reported in Table 6.2, where $N_{u,test}$ is the failure load, δ_u is the axial shortening at the failure load, β_u is the mid-height torsional rotation at the failure load, and Δ_{dM} and Δ_{dm} are respectively the mid-height lateral deflections about the major and minor principal axes at the failure load. Figures 6.5 and 6.6 show the load–mid-height torsional rotation curves for the two series of fixed-ended press-braked stainless steel equal-leg angle section column specimens, respectively. With regard to the failure modes, visible flexural-torsional buckling about the major axis was observed at the mid-height of all the equal-leg angle section columns, with a typical failure specimen A100-700 shown in Figure 6.7. Moreover, as the member length of the specimen increases, flexural buckling about the minor principal axis becomes increasingly visible, indicating the growing interaction of major-axis flexural-torsional buckling with minor-axis flexural buckling; a typical specimen A100-1500 exhibiting this type of interactive buckling mode is shown in Figure 6.8.

Table 6.2: Summary of fixed-ended angle section column test results

Cross-section	Specimen ID	$N_{u,test}$ (kN)	δ_u (mm)	Δ_{dM} (mm)	Δ_{dm} (mm)	β_u (deg)
A 50×50×6	A50-150	244.5	2.13	0.87	0.27	1.89
	A50-150-R	246.5	2.79	2.57	2.13	1.69
	A50-300	227.9	3.70	0.46	0.13	1.69
	A50-360	214.9	2.51	0.46	2.75	0.43
	A50-420	210.6	2.93	0.89	1.61	0.57
	A50-480	197.9	2.48	0.12	2.63	0.32
	A50-540	188.0	2.30	0.03	3.23	0.49
	A50-600	178.6	2.05	1.28	3.52	1.78
A 100×100×6	A100-300	358.2	0.96	0.11	0.22	0.86
	A100-300-R	356.5	0.94	0.03	0.11	1.40
	A100-500	308.7	1.01	0.11	0.25	2.41
	A100-700	306.7	1.28	0.14	0.48	2.86
	A100-900	293.9	1.72	0.03	0.39	2.86
	A100-1100	257.0	1.40	0.29	0.93	2.72
	A100-1300	277.2	1.95	0.51	2.01	6.13
	A100-1500	285.1	2.37	0.28	2.85	4.38

Figure 6.5: Load–mid-height torsional rotation curves for A 50×50×6 specimens

Figure 6.6: Load–mid-height torsional rotation curves for A 100×100×6 specimens

Figure 6.7: Experimental and numerical failure modes for specimen A100-700

Figure 6.8: Experimental and numerical failure modes for specimen A100-1500

6.3 Numerical modelling programme

6.3.1 General

A series of numerical simulations were carried out to supplement the experimental investigation reported in Section 6.2, using the finite element analysis software ABAQUS (ABAQUS, 2012). The FE models were firstly developed and validated against the test results, and then employed to conduct parametric studies to generate further numerical data over a wide range of cross-section dimensions and member lengths. The numerical results were then used together with the experimental data for assessing the accuracy of the current design approaches for fixed-ended press-braked stainless steel equal-leg angle section columns in Section 6.4.

6.3.2 Development of FE models

Shell element S4R has been extensively and successfully used in previous nonlinear numerical simulations of stainless steel open and tubular section members (Zhao et al., 2015, 2016a, 2016b, 2016d; Liang et al., 2019a, 2019c, 2019d, 2020; Sun and Zhao, 2019; Sun et al., 2019; Zhang et al., 2020c, 2021b), and consequently was also adopted in the present finite element modelling. The width and length of the shell element S4R were selected, following a mesh sensitivity study with element size ranging from $0.1t$ to $5t$. For the flat parts of the modelled press-braked equal-leg angle sections, an element size equal to the material thickness was finally adopted, while the corner regions were discretised by a mesh of four elements. The measured (engineering) flat and corner material stress–strain curves for each angle section were first converted into the true stress–strain curves, and then assigned to the respective flat and corner portions of the press-braked angle section column models. The end sections of the finite element models were coupled to two concentric reference points, where modelling of boundary conditions was then carried out. Specifically, the reference point at the loaded end was only allowed to translate longitudinally, while the other reference point was restrained against all degrees of freedom, to achieve the same fixed-ended boundary condition as used in the tests.

Initial geometric imperfections can greatly influence the flexural-torsional buckling behaviour of fixed-ended press-braked stainless steel equal-leg angle section columns, and were thus incorporated into the developed numerical models, in order to accurately replicate the test responses. The initial global geometric imperfection patterns about the two principal axes and the initial torsional geometric imperfection pattern were taken as the respective lowest elastic buckling mode shapes under axial compression, derived through an elastic eigenvalue buckling analysis (ABAQUS, 2012). The measured maximum initial geometric imperfection amplitudes, as listed in Table 6.1, together with another two sets of imperfection values commonly used in previous finite element modelling of fixed-ended carbon steel angle section columns, with the initial torsional geometric imperfection amplitudes equal to $t/(10B)$ and $t/(50B)$ and initial global geometric imperfection amplitudes equal to $L/750$ (Ellobody and Young, 2005; Dinis and Camotim, 2015) about both the principal axes, were used herein to factor the obtained imperfection patterns. Upon incorporation of the initial global and torsional geometric imperfections into the developed numerical models, static Riks analysis (ABAQUS, 2012), accounting for both material and geometric nonlinearities, was conducted to replicate the full range of the experimental load–deformation history for each fixed-ended press-braked stainless steel equal-leg angle section column specimen.

6.3.3 Validation of FE models

Assessment of the accuracy of the developed numerical models was conducted by comparing the failure loads, load–mid-height torsional rotation curves and deformed failure modes obtained from finite element simulations with the corresponding experimental observations. Table 6.3 reports the numerical to test failure load ratios $N_{u,FE}/N_{u,test}$ for the three considered sets of initial global and torsional geometric imperfection combinations, indicating that all three generally result in accurate and consistent predictions of the test failure loads. The test and numerical load–mid-height torsional rotation curves for typical fixed-ended press-braked stainless steel angle section column specimens A100-700 and A100-1500 are compared in Figures 6.9 and 6.10, respectively, where the numerical curves capture well the corresponding experimental responses. Comparisons between the test and numerical failure modes for typical specimens A100-700 exhibiting flexural-torsional buckling and A100-1500

displaying interactive flexural-torsional and flexural buckling are shown in Figures 6.7 and 6.8, respectively, indicating excellent agreement. Overall, the FE models developed in Section 6.3.1 are capable of accurately replicating the test responses of the fixed-ended press-braked stainless steel equal-leg angle section column specimens, and are thus considered to be validated.

Table 6.3: Comparisons of numerical failure loads with test failure loads for varying initial geometric imperfection combinations

Cross-section	Specimen ID	$N_{u,FE}/N_{u,test}$		
		$\omega_{dM} + \omega_{dm} + \omega_{\beta}$	$L/750+L/750+t/(10B)$	$L/750+L/750+t/(50B)$
A 50×50×6	A50-150	0.96	0.86	0.93
	A50-150-R	0.95	0.84	0.91
	A50-300	0.93	0.88	0.93
	A50-360	0.94	0.93	0.94
	A50-420	0.92	0.92	0.93
	A50-480	0.93	0.94	0.96
	A50-540	0.95	1.00	1.00
	A50-600	0.95	1.01	1.01
A 100×100×6	A100-300	1.05	0.99	0.99
	A100-300-R	1.07	0.97	1.01
	A100-500	1.05	0.97	1.00
	A100-700	1.00	0.93	0.94
	A100-900	1.03	0.95	0.94
	A100-1100	1.11	1.01	1.02
	A100-1300	1.04	0.97	0.95
	A100-1500	0.99	0.89	0.90
Mean		0.99	0.94	0.96
COV		0.06	0.06	0.04

Figure 6.9: Experimental and numerical load–mid-height torsional rotation curves for specimen A100-700

Figure 6.10: Experimental and numerical load–mid-height torsional rotation curves for specimen A100-1500

6.3.4 Parametric studies

Parametric studies were carried out in this section using the validated finite element models to generate further numerical data over a wide range of cross-section dimensions and member lengths. In the present parametric studies, the adopted material properties were taken from the corresponding tensile flat and corner coupon tests on A 100×100×6, while the initial torsional geometric imperfection and initial global geometric imperfection along each principal axis were included into the numerical models, with the distribution patterns taken as the respective lowest elastic buckling mode shapes and the amplitudes equal to $t/(50B)$ and $L/750$ (Ellobody and Young, 2005; Dinis and Camotim, 2015), respectively. Regarding the geometric dimensions of the modelled press-braked equal-leg angle sections, the leg widths were fixed at 100 mm, while the section thickness varied between 1.6 mm and 10 mm, leading to a wide spectrum of cross-section B/t ratios from 10.0 to 62.5. For each angle section, 10 member lengths were carefully selected, so that all the columns failed by flexural-torsional buckling. The internal corner radii of the modelled angle sections were set to be 1.5 times the cross-section thicknesses. Table 6.4 summarises the cross-section dimensions and member lengths selected for the present parametric studies. In total, 210 numerical data have been generated through parametric studies.

Table 6.4: Cross-section dimensions and member lengths selected for parametric studies

B (mm)	t (mm)	r (mm)	L (mm)
100	10.0	15.0	300, 450, 600, 750, 900, 1050, 1200, 1350, 1500, 1650
	8.0	12.0	300, 500, 700, 900, 1100, 1300, 1500, 1700, 1900, 2100
	6.6	9.9	300, 550, 800, 1050, 1300, 1550, 1800, 2050, 2300, 2550
	5.7	8.6	300, 600, 900, 1200, 1500, 1800, 2100, 2400, 2700, 3000
	5.0	7.5	300, 650, 1000, 1350, 1700, 2050, 2400, 2750, 3100, 3450
	4.4	6.6	300, 700, 1100, 1500, 1900, 2300, 2700, 3100, 3500, 3900
	4.0	6.0	300, 750, 1200, 1650, 2100, 2550, 3000, 3450, 3900, 4350
	3.6	5.4	300, 800, 1300, 1800, 2300, 2800, 3300, 3800, 4300, 4800
	3.3	5.0	300, 850, 1400, 1950, 2500, 3050, 3600, 4150, 4700, 5250
	3.1	4.7	300, 900, 1500, 2100, 2700, 3300, 3900, 4500, 5100, 5700
	2.9	4.4	300, 950, 1600, 2250, 2900, 3550, 4200, 4850, 5500, 6150
	2.7	4.1	300, 1000, 1700, 2400, 3100, 3800, 4500, 5200, 5900, 6600
	2.5	3.8	300, 1050, 1800, 2550, 3300, 4050, 4800, 5550, 6300, 7050
	2.4	3.6	300, 1100, 1900, 2700, 3500, 4300, 5100, 5900, 6700, 7500
	2.2	3.3	300, 1150, 2000, 2850, 3700, 4550, 5400, 6250, 7100, 7950
	2.1	3.2	300, 1200, 2100, 3000, 3900, 4800, 5700, 6600, 7500, 8400
	2.0	3.0	300, 1250, 2200, 3150, 4100, 5050, 6000, 6950, 7900, 8850
	1.9	2.9	300, 1300, 2300, 3300, 4300, 5300, 6300, 7300, 8300, 9300
	1.8	2.7	300, 1350, 2400, 3450, 4500, 5550, 6600, 7650, 8700, 9750
	1.7	2.6	300, 1450, 2600, 3750, 4900, 6050, 7200, 8350, 9500, 10650
	1.6	2.4	300, 1500, 2700, 3900, 5100, 6300, 7500, 8700, 9900, 11100

6.4 Assessment of current design codes and other approaches

6.4.1 General

In this section, the existing design rules for fixed-ended press-braked stainless steel equal-leg angle section columns, as given in three design codes – EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) and other recently proposed design approaches (Dinis and Camotim, 2015), are described and analysed. Quantitative assessment of each design method is then made through comparing the test and numerical failure loads N_u against the unfactored design resistance predictions $N_{u,pred}$. Table 6.5 reports the mean $N_u/N_{u,pred}$ ratio for each design method.

Table 6.5: Comparisons of test and numerical failure loads with predicted resistances from different design methods

No. of tests: 16 No. of FE simulations: 210	$N_u/N_{u,EC3}$	$N_u/N_{u,ASCE}$	$N_u/N_{u,AS/NZS}$	$N_u/N_{u,DSM}$
Mean	1.89	2.39	2.39	1.02
COV	0.54	0.47	0.47	0.10

6.4.2 EN 1993-1-4

The current European code EN 1993-1-4 (CEN, 2015) uses buckling curves for the design of stainless steel open and tubular section columns susceptible to global buckling. For press-braked equal-leg angle section members in compression, the EC3 flexural-torsional buckling resistance is given by Eq. (6.6),

$$N_{u,EC3} = \begin{cases} \chi_{FT} A \sigma_{0.2} & \text{for non - slender sections} \\ \chi_{FT} A_{eff} \sigma_{0.2} & \text{for slender sections} \end{cases} \quad (6.6)$$

where $A \sigma_{0.2}$ or $A_{eff} \sigma_{0.2}$ is defined as the cross-section local buckling resistance, and χ_{FT} is the reduction factor, which defines the EC3 design column flexural-torsional buckling curve and can be calculated from Equation (6.7),

$$\chi_{FT} = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{FT}^2}} \leq 1 \quad (6.7)$$

where ϕ is a buckling coefficient, given by Equation (6.8), and $\bar{\lambda}_{FT}$ is the non-dimensional flexural-torsional buckling slenderness, as determined from Equation (6.9),

$$\phi = 0.5 \left[1 + \alpha_{EC3} (\bar{\lambda}_{FT} - \bar{\lambda}_0) + \bar{\lambda}_{FT}^2 \right] \quad (6.8)$$

$$\bar{\lambda}_{FT} = \begin{cases} \sqrt{\frac{A \sigma_{0.2}}{N_{cr,FT}}} & \text{for non - slender sections} \\ \sqrt{\frac{A_{eff} \sigma_{0.2}}{N_{cr,FT}}} & \text{for slender sections} \end{cases} \quad (6.9)$$

where α_{EC3} is the imperfection factor and equal to 0.34 for stainless steel columns susceptible to flexural-torsional buckling, $\bar{\lambda}_0 = 0.2$ is the limiting slenderness, and $N_{cr,FT} = A f_{crft}$ is the elastic critical flexural-torsional buckling load for equal-leg angle section columns, where f_{crft} is the elastic critical flexural-torsional buckling stress, and may be determined numerically by means of the finite element software CUFSM (Schafer and Adány, 2006) and GBTUL (Bebiano et al., 2018), or calculated approximately using simplified formulations (Dinis and Camotim, 2015). Note that the elastic critical flexural-torsional buckling stress f_{crft} is relatively insensitive to the member length, but mainly dependent on the dimensions of the angle section. Thus, the corresponding non-dimensional flexural-torsional buckling slenderness $\bar{\lambda}_{FT}$ remains almost the same for equal-leg angle section columns with the same cross-section size but different member lengths.

The accuracy of the EC3 design provisions for fixed-ended press-braked stainless steel equal-leg angle section columns was assessed through comparisons against the test and numerical failure loads. As shown in Table 6.5, the mean test (or numerical) to EC3 predicted failure load ratio is equal to 1.89, with the corresponding COV of 0.54, indicating that the European code EN 1993-1-4 (CEN, 2015) results in unduly conservative and scattered flexural-torsional buckling resistance predictions for fixed-ended press-braked stainless steel equal-leg angle section columns. The test and numerical failure loads are also normalised by the corresponding cross-section local buckling resistances and plotted against the non-dimensional flexural-torsional buckling slendernesses $\bar{\lambda}_{FT}$ in Figure 6.11, together with the EC3 design column flexural-torsional buckling curve. It is evident that the test and numerical data points with similar non-dimensional slenderness $\bar{\lambda}_{FT}$, corresponding to angle section columns with the same cross-section size but different member lengths, show a high degree of scatter, and thus a single design curve cannot fully capture their structural behaviour and load-carrying resistances. Therefore, length-dependent design curves are generally required in order to account for the influence of member length on the flexural-torsional buckling resistances of fixed-ended press-braked stainless steel equal-leg angle section columns.

Figure 6.11: Comparisons of test and numerical results with EN 1993-1-4 design column flexural-torsional buckling curve

6.4.3 SEI/ASCE 8-02 and AS/NZS 4673

The current SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) employ the same design rules for press-braked stainless steel equal-leg angle section columns, of which the key principle lies in the consideration of an additional loading eccentricity to the minor principal axis of $L/1000$, according to Clause 3.4 of SEI/ASCE 8-02 (ASCE, 2002) and Clause 3.4.1 of AS/NZS 4673 (AS/NZS, 2001). Therefore, SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) require the angle section columns subjected to concentric compression to be designed as beam-columns under eccentric compression, based on the interaction formula given by Equation (6.10),

$$\frac{N_{u,ASCE}}{N_n} + \frac{C_{mz} M_{uz,ASCE}}{\alpha_{nz} M_{nz}} = 1 \quad \text{or} \quad \frac{N_{u,AS/NZS}}{N_n} + \frac{C_{mz} M_{uz,AS/NZS}}{\alpha_{nz} M_{nz}} = 1 \quad (6.10)$$

where $N_{u,ASCE}$ (or $N_{u,AS/NZS}$) is the design compression load, $M_{uz,ASCE} = N_{u,ASCE} L / 1000$ (or $M_{uz,AS/NZS} = N_{u,AS/NZS} L / 1000$) is the design bending moment about the minor principal axis, resulting from the initial design loading eccentricity of $L/1000$, N_n is the member nominal

compression resistance, defined as the minimum of the critical flexural, torsional, and flexural-torsional buckling loads, M_{nz} is the cross-section bending moment resistance about the minor principal axis, as determined based on Clause 3.3.1 of SEI/ASCE 8-02 (ASCE, 2002) (or Clause 3.3.3 of AS/NZS 4673 (AS/NZS, 2001)), C_{mz} is the equivalent uniform moment factor, with the value equal to unity for beam-columns with equal end moments, and $\alpha_{nz}=1/(1-N_{u,ASCE}/N_{cr})$ (or $\alpha_{nz}=1/(1-N_{u,AS/NZS}/N_{cr})$) is the magnification factor to account for the second-order effect, where N_{cr} is the Euler buckling load about the minor principal axis.

The accuracy of the SEI/ASCE 8-02 (ASCE, 2002) (or AS/NZS 4673 (AS/NZS, 2001)) flexural-torsional buckling resistance predictions was assessed, based on the obtained test and numerical failure loads. The mean $N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$) ratio, as reported in Table 6.5, is equal to 2.39, with the corresponding COV of 0.47, revealing that SEI/ASCE 8-02 (ASCE, 2002) (or AS/NZS 4673 (AS/NZS, 2001)) yields even more conservative but slightly less scattered flexural-torsional buckling resistance predictions than EN 1993-1-4 (CEN, 2015). This can also be seen from Figure 6.12, where the test (or numerical) to predicted failure load ratios $N_u/N_{u,pred}$ for each design code are plotted against the member non-dimensional flexural-torsional buckling slendernesses $\bar{\lambda}_{FT}$.

Figure 6.12: Comparisons of test and numerical results with EN 1993-1-4 and SEI/ASCE 8-02 (or AS/NZS 4673) resistance predictions

6.4.4 DSM-based design approach

Silvestre et al. (2013), Dinis et al. (2012) and Mesacasa Jr. et al. (2014) numerically studied the buckling and post-buckling behaviour of carbon steel equal-leg angle section columns, and highlighted (i) the necessity of adopting length-dependent design curves for flexural-torsional buckling and (ii) the existence of strong interaction of flexural-torsional buckling with minor-axis flexural buckling. Novel design approaches for carbon steel equal-leg angle section columns, accounting for both the length-dependence of flexural-torsional buckling behaviour and the interactive flexural-torsional and flexural buckling behaviour, were subsequently proposed by Landesmann et al. (2017) and Dinis and Camotim (2015), based on the direct strength method (Schafer, 2008), with the design expression given by Equation (6.11),

$$N_{u,DSM} = \begin{cases} Af_{ne} & \text{for } \lambda_{fte} \leq (0.5 + \sqrt{0.25 - b'})^{\frac{1}{2a'}} \\ Af_{ne} \frac{1}{\lambda_{fte}^{2a'}} \left(1 - \frac{b}{\lambda_{fte}^{2a'}} \right) & \text{for } \lambda_{fte} > (0.5 + \sqrt{0.25 - b'})^{\frac{1}{2a'}} \end{cases} \quad (6.11)$$

where $N_{u,DSM}$ is the design flexural-torsional buckling resistance, a' and b' are the length-dependent coefficients, as determined from Equations (6.12) and (6.13), respectively, where Δ_f is a measure of length-dependence, as given by Equation (6.14), where f_{bt} is the elastic critical torsional buckling stress and f_{crft} is the elastic critical flexural-torsional buckling stress, and λ_{fte} is the ‘interactive slenderness’, taking due account of the interaction of flexural-torsional buckling behaviour and flexural buckling behaviour, and determined as $\sqrt{f_{ne} / f_{crft}}$, where f_{ne} is the design flexural buckling stress about the minor principal axis, as determined by Equation (6.15), where $\lambda_c = \sqrt{f_y / f_{cre}}$ is calculated as the square root of the ratio of the yield stress f_y to the elastic critical flexural buckling stress about the minor principal axis f_{cre} .

$$a' = \begin{cases} 0.001\Delta_f^3 - 0.032\Delta_f^2 + 0.25\Delta_f + 0.4 & \text{for } \Delta_f \leq 5 \\ 0.001\Delta_f + 0.97 & \text{for } \Delta_f > 5 \end{cases} \quad (6.12)$$

$$b' = \begin{cases} 0.014\Delta_f + 0.15 & \text{for } \Delta_f \leq 7 \\ 0.248 & \text{for } \Delta_f > 7 \end{cases} \quad (6.13)$$

$$\Delta_f = \frac{f_{bt} - f_{cft}}{f_{bt}} \times 100 \quad (6.14)$$

$$f_{ne} = \begin{cases} f_y(0.658^{\lambda_c^2}) & \text{for } \lambda_c \leq 1.5 \\ f_y\left(\frac{0.877}{\lambda_c^2}\right) & \text{for } \lambda_c > 1.5 \end{cases} \quad (6.15)$$

The DSM-based design approach (Dinis and Camotim, 2015; Landesmann et al., 2017) was shown to result in much more accurate and consistent flexural-torsional buckling resistance predictions for fixed-ended carbon steel equal-leg angle section columns, while its applicability to the design of fixed-ended press-braked stainless steel equal-leg angle section columns is assessed herein. As reported in Table 6.5, the mean ratio of the test (or numerical) failure loads to the predicted resistances from the DSM-based design approach $N_u/N_{u,DSM}$ is equal to 1.02, with the corresponding COV of 0.10, indicating a substantially high level of improvement in the flexural-torsional buckling resistance predictions of fixed-ended press-braked stainless steel angle section columns over the established international design standards. Graphical comparisons between the DSM-based design approach and the current design codes are also shown in Figures 6.13 and 6.14, where the ratios of the test (or numerical) failure loads to the predicted resistances from each design method $N_u/N_{u,pred}$ are plotted against the interactive slendernesses λ_{fe} . In comparison with the existing design codes, which lead to unduly conservative and scattered flexural-torsional buckling resistance predictions, particularly for angle section columns with large interactive slendernesses λ_{fe} , the new DSM-based design approach was generally found to yield accurate and consistent resistance predictions across the full range of λ_{fe} . However, it should also be noted that in spite of the overall accuracy of the new DSM-based design proposal, a large portion of the predicted resistances are still on the unsafe side (i.e. the ratio of $N_u/N_{u,DSM}$ is less than unity). This can be principally attributed to the fact that the DSM-based design approach (Dinis and Camotim, 2015) was originally developed for carbon steel equal-leg angle section columns, without explicitly taking into account the nonlinear material characteristics of stainless steel.

Figure 6.13: Comparisons of test and numerical results with EN 1993-1-4 and DSM resistance predictions

Figure 6.14: Comparisons of test and numerical results with SEI/ASCE 8-02 (or AS/NZS 4673) and DSM resistance predictions

6.4.5 Summary

Summing up the comparison results, it was generally found that the American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) lead to the most conservative flexural-torsional buckling resistance predictions for fixed-ended press-braked stainless steel equal-leg angle section columns. The European code EN 1993-1-4 (CEN, 2015) offers slightly more accurate resistance predictions but with a similar high level of scatter, compared with SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). The DSM-based design approach takes into due account of the length-dependence of flexural-torsional buckling behaviour and the interaction of flexural-torsional buckling behaviour and flexural buckling behaviour, and was shown to result in substantially improved resistance predictions for fixed-ended press-braked stainless steel equal-leg angle section columns on average. However, since the DSM-based design approach (Dinis and Camotim, 2015) was originally developed for carbon steel equal-leg angle section columns, without explicitly accounting for the nonlinear material characteristics of stainless steel, a large portion of the resistance predictions are unsafe. More suitable design approaches for press-braked stainless steel equal-leg angle section columns susceptible to flexural-torsional buckling are generally necessary.

Quantitative assessment of the current design standards and the DSM-based design proposal was also conducted, based on the test results only. Table 6.6 reports the test to predicted failure load ratios for the four considered design methods $N_{u,test}/N_{u,pred}$, revealing that the three design standards lead to conservative flexural-torsional buckling resistance predictions, while the DSM proposal provides a high level of design accuracy on average, but with many overestimated predicted resistances.

Table 6.6: Comparisons of test failure loads with predicted resistances

Cross-section	Specimen ID	$N_{u, test}/N_{u, EC3}$	$N_{u, test}/N_{u, ASCE}$	$N_{u, test}/N_{u, AS/NZS}$	$N_{u, test}/N_{u, DSM}$
A 50×50×6	A50-150	1.42	1.71	1.71	1.32
	A50-150-R	1.43	1.73	1.73	1.33
	A50-300	1.39	1.85	1.85	1.25
	A50-360	1.31	1.80	1.80	1.19
	A50-420	1.28	1.80	1.80	1.16
	A50-480	1.23	1.76	1.76	1.12
	A50-540	1.15	1.69	1.69	1.06
	A50-600	1.11	1.67	1.67	1.03
A 100×100×6	A100-300	1.59	1.53	1.53	0.89
	A100-300-R	1.49	1.44	1.44	0.84
	A100-500	1.50	1.42	1.42	0.86
	A100-700	1.57	1.49	1.49	0.91
	A100-900	1.54	1.50	1.50	0.92
	A100-1100	1.40	1.39	1.39	0.85
	A100-1300	1.50	1.53	1.53	0.95
	A100-1500	1.54	1.62	1.62	1.01
Mean		1.40	1.62	1.62	1.04
COV		0.11	0.09	0.09	0.16

6.5 Proposed design method

6.5.1 Methodology to overcome shortcomings in existing design methods

Limitations reflected by existing design methods include: (i) underestimation of the length-dependent characteristic of flexural-torsional buckling resistances, (ii) ignorance of interactive failure modes, i.e. the interaction between the major-axis flexural-torsional buckling and minor-axis flexural buckling, and (iii) failure to consider the nonlinear material property of stainless steel, i.e. the gradual yielding behaviour. Improved design method, where the material nonlinearity is incorporated, is then proposed for fixed-ended press-braked stainless steel angle section columns failing by flexural-torsional buckling, based on the DSM-based design approach developed by Dinis and Camotim (2015) for press-braked angle section columns made of carbon steel. The derivation procedure of the new design method may be summarised as follows:

(i) The first step, as described in Section 6.5.2, is to propose a series of length-dependent flexural-torsional strength curves for press-braked stainless steel angle section columns. This first requires to obtain a fairly large numerical failure load data bank on press-braked stainless steel angle section columns failing by pure flexural-torsional buckling. In the present numerical modelling programme, the leg widths of the modelled angle section columns were kept constant as 100 mm, while the wall thicknesses varied between 1.6 mm and 10 mm, resulting in 21 angle sections and a wide spectrum of cross-section B/t ratios from 10.0 to 62.5 being considered. For each angle section, 10 member lengths were selected so that 10 Δ_f values were obtained: 0.2, 0.5, 1, 2, 3, 4, 5, 6, 7, 8. For each Δ_f value, therefore, 21 pure flexural-torsional buckling failure load data can be obtained and the expressions for parameters a' and b' in terms of Δ_f can then be sought through the curve fitting technique. A series of flexural-torsional strength curves can be subsequently defined for press-braked stainless steel angle section columns with different member lengths.

(ii) The global strength curve for stainless steel angle section columns was then carefully selected based on the numerically derived flexural buckling failure loads, as reported in Section 6.5.3. With regard to the cross-section dimensions of the modelled equal-leg angle sections, three outer leg widths equal to 80 mm, 100 mm and 120 mm were selected, with the wall thicknesses ranging from 9 mm to 18 mm. A total of nine equal-leg angle sections were modelled in the present numerical simulations; the modelled equal-leg angle sections covered all three classes of non-slender cross-sections (i.e. Class 1, 2 and 3) according to the slenderness limits in EN 1993-1-4 (CEN, 2015), and were also defined as non-slender cross-sections according to the limiting width-to-thickness ratios in SEI/ASCE 8-02 (ASCE, 2002), ensuring that no local plate buckling occurs and the columns fail by pure flexural buckling about the minor principal axis. For each modelled equal-leg angle section, twenty effective member lengths ranging from 300 mm to 6100 mm were examined to cover a wide member slenderness $\lambda_c = \sqrt{\sigma_{0.2}/f_{cre}}$ between 0.33 and 4.50.

(iii) Following the strategy adopted in the current DSM design against interactive local–global buckling failure, the design of interactive flexural-torsional–minor-axis flexural buckling failure for angle section columns was performed through combining the newly

developed flexural-torsional strength curves with the selected global strength curve and is described in Section 6.5.4.

6.5.2 Derivation of flexural-torsional strength curves

The pure flexural-torsional buckling failure load data bank with the material nonlinearity of stainless steel being considered was firstly obtained by the finite element analysis software ABAQUS (ABAQUS, 2012). In this section, the adopted modelling procedures and techniques were the same as those described in Section 6.3, except for that the minor-axis flexure of each angle section column FE model is continuously restrained along the column length. The adopted boundary conditions are shown in Figure 6.15; this ensures that all the modelled press-braked stainless steel angle section columns suffer from pure flexural-torsional buckling failure. Moreover, the initial geometric imperfection distribution along the column length was taken as the first critical elastic flexural-torsional buckling mode shape, which can be derived through an elastic eigenvalue buckling analysis (ABAQUS, 2012), and the imperfection magnitude was set to be equal to $L/1000$ (Dinis and Camotim, 2015).

Figure 6.15: Boundary conditions for pure flexural-torsional buckling

The numerically obtained pure flexural-torsional buckling failure loads $N_{u,FE}$, normalised by the yield loads $A\sigma_{0.2}$, are plotted against λ_{ft} , defined as $\sqrt{\sigma_{0.2} / f_{crft}}$, in Figure 6.16, where the DSM local strength curve is also shown. The comparison results generally indicated great vertical dispersion of the failure loads for the same angle section size but with different column lengths. Moreover, the majority of the data points are observed to be located below the DSM local strength curve, indicating unsafe predictions of the flexural-torsional buckling resistances by this curve. This demonstrates that a single design curve is not capable of accurately predicting the flexural-torsional buckling resistances of angle section columns of different member lengths and development of a series of length-dependent flexural-torsional strength curves is necessary.

Figure 6.16: Comparisons of numerically derived flexural-torsional buckling failure loads with DSM local strength curve

It can be seen from Figure 6.16 that the data points, regardless of the angle section size, generally follow the same trend as the DSM local strength curve. Therefore, similar to the equation format of DSM local strength curve, flexural-torsional strength curves for stainless steel angle section columns were proposed by Equation (6.16), where $N_{u,nft}$ is the design pure flexural-torsional buckling resistance, a' and b' are parameters related to the column length to capture the previously mentioned vertical dispersion. The column length effect is reflected by the parameter Δ_f , as previously defined by Equation (6.14), representing the percentage drop

of the elastic critical flexural-torsional buckling stress for a specific column length compared with the cross-section elastic critical torsional buckling stress. For each combination of a' and b' , which are expressed in terms of Δ_f , a flexural-torsional strength curve is defined correspondingly, through which the length-independent characteristic of the column flexural-torsional buckling resistances can be captured. Following a careful regression fitting procedure of Equation (6.16) to the numerically derived pure flexural-torsional buckling failure loads, a set of a' and b' values could be found for different Δ_f values and the final expressions are given by Equations (6.17) and (6.18), respectively.

$$N_{u, nft} = \begin{cases} A\sigma_{0.2} & \text{for } \lambda_{ft} \leq (0.5 + \sqrt{0.25 - b'})^{\frac{1}{2a'}} \\ A\sigma_{0.2} \frac{1}{\lambda_{ft}^{2a'}} \left[1 - \frac{b'}{\lambda_{ft}^{2a'}}\right] & \text{for } \lambda_{ft} > (0.5 + \sqrt{0.25 - b'})^{\frac{1}{2a'}} \end{cases} \quad (6.16)$$

$$a' = \begin{cases} -0.08\Delta_f + 0.525 & \text{for } \Delta_f \leq 1 \\ 0.04\Delta_f + 0.486 & \text{for } \Delta_f > 1 \end{cases} \quad (6.17)$$

$$b' = \begin{cases} 0.028\Delta_f + 0.222 & \text{for } \Delta_f \leq 1 \\ 0.25 & \text{for } \Delta_f > 1 \end{cases} \quad (6.18)$$

Comparisons of the numerically derived pure flexural-torsional buckling failure loads $N_{u, FE}$, normalised by the yield loads $A\sigma_{0.2}$, with the newly proposed flexural-torsional strength curves as well as the flexural-torsional strength curves proposed by Dinis and Camotim (2015) for carbon steel angle section columns for different Δ_f values are shown in Figures 6.17(a)–6.17(j), respectively. The graphical comparison results generally indicated that the vertical dispersion of column flexural-torsional buckling resistances due to increasing column lengths can be well captured by the newly proposed flexural-torsional strength curves. Moreover, the proposed design curves, with the distinctive material nonlinearity of stainless steel being considered during the derivation process, were also observed to be capable of predicting the flexural-torsional failure loads accurately over the whole considered range of non-dimensional flexural-torsional buckling slenderness λ_{ft} , in contrast to the unsafe predictions of resistances for press-braked stainless steel angle section columns with relatively small slenderness values and overly conservative resistance predictions for press-braked stainless

steel angle section columns with relatively large slenderness values by those curves proposed by Dinis and Camotim (2015).

(i) $\Delta_f = 7.0$

(j) $\Delta_f = 8.0$

Figure 6.17: Comparisons of numerically derived flexural-torsional buckling failure loads with newly proposed flexural-torsional strength curves for different Δ_f values

6.5.3 Global strength curve

Global strength curve that is able to accurately predict the flexural buckling resistances of press-braked stainless steel angle section columns about their minor axes is then sought in this section. Three global strength curves were considered: (i) the DSM global strength curve (Schafer, 2008), as defined by Equation (6.14), (ii) the EC3 flexural buckling curve for open sections with the imperfection coefficient $\alpha_{EC3} = 0.49$ and plateau length $\bar{\lambda}_0 = 0.4$ (CEN, 2015) and (iii) a revised DSM global buckling curve with the expression given by Equation (6.19). Comparisons of these three global strength curves with the normalised pure minor-axis flexural buckling failure load data derived from numerical modelling $N_{u,FE}/A\sigma_{0.2}$ are shown in Figure 6.18. The revised DSM global strength curve is shown to provide accurate, consistent and safe-sided minor-axis flexural buckling resistance predictions for press-braked stainless steel angle section columns, in contrast to many unsafe results obtained by the current DSM global strength design curve (Schafer, 2008) for stainless steel angle section columns with relatively small and intermediate member non-dimensional slenderness λ_c , and EC3 flexural buckling curve (CEN, 2015) for those long stainless steel angle section columns.

$$f_{ne} = \begin{cases} \sigma_{0.2}(0.508^{\lambda_c^{1.64}}) & \text{for } \lambda_c \leq 1.5 \\ \sigma_{0.2}\left(\frac{0.603}{\lambda_c^{2.09}}\right) & \text{for } \lambda_c > 1.5 \end{cases} \quad (6.19)$$

Figure 6.18: Comparisons of numerical failure loads with DSM, EN 1993-1-4 and proposed global strength curves

6.5.4 Improved DSM for fixed-ended stainless steel angle section columns

The last step moves towards the consideration of interaction between flexural-torsional buckling and minor-axis flexural buckling, as highlighted in Section 6.5.1. Similar to the strategy adopted in current DSM against the interactive local–global buckling failure, interaction between the major-axis flexural-torsional buckling and minor-axis flexural buckling is considered through replacing the 0.2% proof stress $\sigma_{0.2}$ in Equation (6.16) by the minor-axis flexural buckling stress f_{ne} determined from the revised DSM global strength curve, defined by Equation (6.19), resulting in the improved DSM resistance predictions $N_{u,p}$ for fixed-ended press-braked stainless steel angle section columns susceptible to flexural-torsional buckling failure, as given by Equation (6.20).

$$N_{u,p} = \begin{cases} Af_{ne} & \text{for } \lambda_{fte} \leq (0.5 + \sqrt{0.25 - b'})^{\frac{1}{2a'}} \\ Af_{ne} \frac{1}{\lambda_{fte}^{2a'}} \left[1 - \frac{b'}{\lambda_{fte}^{2a'}} \right] & \text{for } \lambda_{fte} > (0.5 + \sqrt{0.25 - b'})^{\frac{1}{2a'}} \end{cases} \quad (6.20)$$

6.5.5 Assessment of proposed method and reliability analysis

The accuracy of the proposed design method for fixed-ended stainless steel angle section columns susceptible to flexural-torsional buckling was assessed through comparisons of the predicted resistances with the test and numerical failure loads. As reported in Table 6.7, the mean ratio of test and numerical failure loads to the predicted resistances by the proposed design method is equal to 1.10, with the corresponding COV of 0.06, revealing that the newly proposed design method yields accurate and consistent predictions of flexural-torsional buckling resistance for fixed-ended press-braked stainless steel angle section columns. This is also evidenced by the graphical assessment results, as shown in Figure 6.19, where the test and numerical failure loads N_u are normalised by the resistance predictions from the newly proposed design method $N_{u,p}$ and then plotted against the non-dimensional flexural-torsional buckling slendernesses λ_{ft} . In comparison with the existing design codes (CEN, 2015; ASCE, 2002; AS/NZS, 2001), the newly proposed design method is shown to yield a significantly higher level of accuracy and consistency in the flexural-torsional buckling resistance predictions for fixed-ended press-braked stainless steel angle section columns, owing to the rational consideration of the length-dependent characteristic of flexural-torsional buckling and the interaction of flexural-torsional buckling with minor-axis flexural buckling. Moreover, since the material nonlinearity of stainless steel was incorporated during the derivation of flexural-torsional strength curves, the new proposal is also found to lead to safe-sided resistance predictions for angle section columns over the whole examined slenderness range, in contrast to many unsafe results given by the DSM-based design approach originally proposed by Dinis and Camotim (2015) for angle section columns made of carbon steel.

Table 6.7: Comparisons of test and numerical failure loads with predicted resistances

No. of tests: 16 No. of FE simulations: 210	$N_u/N_{u,EC3}$	$N_u/N_{u,ASCE}$	$N_u/N_{u,AS/NZS}$	$N_u/N_{u,DSM}$	$N_u/N_{u,p}$
Mean	1.89	2.39	2.39	1.02	1.10
COV	0.54	0.47	0.47	0.10	0.06

Figure 6.19: Comparisons of test and numerical results with resistance predictions from newly proposed design method

The reliability of the newly proposed design method was assessed by means of statistical analyses, based on the requirements as stated in EN 1990 (CEN, 2002). The key statistical parameters for press-braked stainless steel angle section columns according to EN 1990 (CEN, 2002) are reported in Table 6.8, where γ_{M1} is the partial safety factor for member resistance. The resulting partial resistance factor γ_{M1} for the proposed design method, as shown in Tables 6.8, is equal to 0.97, which is less than currently adopted value 1.1 in EN 1993-1-4 (CEN, 2015), demonstrating the reliability of the design method proposed in Section 6.5 for fixed-ended press-braked stainless steel angle section columns failing by flexural-torsional buckling.

Table 6.8: Reliability analysis results calculated according to EN 1990

Design method	No. of test and FE data	$k_{d,n}$	b	V_δ	V_r	γ_{M1}
DSM	226	3.130	0.937	0.085	0.116	1.18
New proposal	226	3.130	1.070	0.057	0.096	0.97

6.5.6 Worked example

A worked example on the test specimen A100-700 is presented in this section to demonstrate the design procedure of the newly proposed design method for fixed-ended press-braked stainless steel angle section columns. The cross-section geometric dimensions and weighted material properties against the cross-section area are as follows: leg width $B=99.8$ mm, wall thickness $t=5.89$ mm, corner internal radius $r=9$ mm, column length $L=700.2$ mm, cross-section area $A=1110.8$ mm², Young's modulus $E=197932$ MPa, shear modulus $G=76128$ MPa, 0.2% proof stress $\sigma_{0.2}=397$ MPa.

(1) Determine parameter Δ_f

The elastic torsional, major-axis flexural, and flexural-torsional buckling stresses can be calculated approximately according to the simplified equations (Dinis and Camotim, 2015; Landesmann et al., 2017) as:

$$f_{bt} = G \frac{t^2}{B^2} + \pi^2 \frac{Et^2}{12(L/2)^2} = 76128 \times \frac{5.89^2}{99.8^2} + \pi^2 \frac{197932 \times 5.89^2}{12 \times (700.2/2)^2} = 328 \text{ MPa}$$

$$f_{bf} = \pi^2 \frac{EB^2}{6(L/2)^2} = \pi^2 \frac{197932 \times 100.0^2}{6 \times (700.2/2)^2} = 24919 \text{ MPa}$$

$$\begin{aligned} f_{crft} &= 0.8(f_{bt} + f_{bf} - \sqrt{(f_{bt} + f_{bf})^2 - 2.5f_{bt}f_{bf}}) \\ &= 0.8 \times (328 + 24919 - \sqrt{(328 + 24919)^2 - 2.5 \times 328 \times 24919}) = 326 \text{ MPa} \end{aligned}$$

$$\text{Then, } \Delta_f = \frac{f_{bt} - f_{crft}}{f_{bt}} \times 100 = \frac{328 - 326}{328} \times 100 = 0.49 < 1.0$$

(2) Determine the design flexural buckling stress about the cross-section minor principal axis f_{ne}

The whole section is effective since local buckling is fully restrained as described in Section 6.5.3. The elastic critical flexural buckling stress about minor principal axis is:

$$f_{cre} = \pi^2 \frac{EB^2}{24(L/2)^2} = \pi^2 \frac{197932 \times 100.0^2}{24 \times (700.2/2)^2} = 6230 \text{ MPa}$$

Non-dimensional slenderness:

$$\lambda_c = \sqrt{\frac{\sigma_{0.2}}{f_{cre}}} = \sqrt{\frac{397}{6230}} = 0.25 < 1.5$$

Then the design flexural buckling stress about minor principal axis is:

$$f_{ne} = \sigma_{0.2} (0.508^{\lambda_c^{1.64}}) = 397 \times 0.508^{0.25^{1.64}} = 370 \text{ MPa}$$

(3) Determine the design compression resistance $N_{u,p}$

$$a' = -0.08\Delta_f + 0.525 = -0.08 \times 0.49 + 0.525 = 0.486$$

$$b' = 0.028\Delta_f + 0.222 = 0.028 \times 0.49 + 0.222 = 0.236$$

Interactive slenderness:

$$\lambda_{fie} = \sqrt{f_{ne} / f_{cft}} = \sqrt{370 / 326} = 1.07 > (0.5 + \sqrt{0.25 - b'})^{\frac{1}{2a'}} = 0.61$$

Then, the design compression resistance of the angle section column specimen is:

$$N_{u,p} = Af_{ne} \frac{1}{\lambda_{fie}^{2a'}} \left(1 - \frac{b'}{\lambda_{fie}^{2a'}} \right) = 1110.8 \times 370 \times \frac{1}{1.07^{2 \times 0.486}} \times \left(1 - \frac{0.236}{1.07^{2 \times 0.486}} \right) = 300.8 \text{ kN}$$

$$N_{u,test}/N_{u,p}=1.02; N_{u,test}/N_{u,EC3}=1.57; N_{u,test}/N_{u,ASCE}=N_{u,test}/N_{u,AS/NZS}=1.49; N_{u,test}/N_{u,DSM}=0.91$$

6.6 Concluding remarks

The structural behaviour and resistances of fixed-ended press-braked stainless steel equal-leg angle section columns susceptible to flexural-torsional buckling were experimentally and numerically investigated in this chapter. The experimental programme was carried out on two press-braked austenitic stainless steel equal-leg angle sections, and included material tensile coupon tests, initial geometric imperfection measurements and a total of sixteen fixed-ended column tests. The obtained test data were adopted in a parallel FE simulation programme for the validation of numerical models, which were then employed to perform parametric studies to generate further numerical data on fixed-ended press-braked stainless steel equal-leg angle section columns over a wide range of cross-section dimensions and member lengths. The obtained test and numerical results were carefully studied, and used to assess the accuracy of the existing design rules, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). The assessment results generally indicated an unduly high level of conservatism and scatter of the codified design flexural-torsional buckling resistance predictions. The applicability of the DSM-based design method for carbon steel equal-leg angle section columns (Dinis and Camotim, 2015) to the design of their press-braked stainless steel counterparts was also assessed, indicating that the new proposal substantially improves the design accuracy over the existing design standards, but with many unsafe resistance predictions. Finally, a new design method, involving the use of a set of newly derived length-dependent flexural-torsional strength curves, where the nonlinear material property of stainless steel was taken into due account, and a revised DSM global strength curve, was proposed and shown to provide accurate, consistent and safe-sided resistance predictions for fixed-ended press-braked stainless steel angle section columns. Statistical analyses were also conducted according to EN 1990 (CEN, 2002) and demonstrated the reliability of the new proposal.

CHAPTER 7

FLEXURAL BUCKLING BEHAVIOUR OF CHANNEL SECTION COLUMNS

7.1 Introduction

Chapters 3–5 have studied the local buckling behaviour and resistances of press-braked stainless steel channel sections under isolated loading and combined loading, while the focus of this chapter is on the structural behaviour of press-braked stainless steel channel section columns failing by flexural buckling about the minor principal axis. In this chapter, an experimental programme is firstly conducted, and includes initial global and local geometric imperfection measurements and pin-ended column tests on twelve press-braked stainless steel channel section column specimens. The experimental programme is supplemented by a numerical modelling programme, including a validation study, where finite element models are developed and validated against the test results, and a parametric study, where the validated finite element models are used to generate further numerical data on press-braked stainless steel channel section columns over a wide range of cross-section dimensions and

member effective lengths. The obtained test and numerical data are used to assess the accuracy of the codified design rules, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002), AS/NZS 4673 (AS/NZS, 2001) and SCI P413 (SCI, 2017), for press-braked stainless steel channel section columns failing by flexural buckling about the minor principal axis. The findings of this research have been reported by Zhang et al. (2020e).

7.2 Testing programme

7.2.1 Test specimens

Two channel sections C 80×50×6 and C 120×60×6, press-braked from grade EN 1.4420 austenitic stainless steel sheets, were adopted in the testing programme, and both of the two adopted channel sections are categorised as Class 3 based on the slenderness limits given in EN 1993-1-4 (CEN, 2015) and also fall in the category of non-slender cross-sections specified in SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). For each adopted channel section, six columns were prepared with different member lengths, resulting in a total of twelve column specimens in the testing programme. The identifier of each specimen includes the cross-section label ‘C1’ or ‘C2’ (‘C1’ signifying C 80×50×6 and ‘C2’ representing C 120×60×6), a letter ‘L’ (indicating length) and a number (for the purpose of differentiating specimens with different member lengths but the same cross-section size), e.g., C1-L1. The measured geometric dimensions of each press-braked stainless steel channel section column specimen are reported in Table 7.1.

Table 7.1: Measured geometric dimensions and initial geometric imperfections of press-braked stainless steel channel section column specimens

Cross-section	Specimen ID	L (mm)	L_e (mm)	H (mm)	B (mm)	t (mm)	r (mm)	ω_0 (mm)	ω_g (mm)	$\bar{\lambda}_z$
C 80×50×6	C1-L1	449.8	599.8	80.08	49.97	5.96	8.94	0.03	-0.40	0.54
	C1-L2	598.9	748.9	80.11	49.89	5.95	8.93	0.03	-0.49	0.68
	C1-L3	750.2	900.2	79.94	50.06	5.96	8.94	0.03	-0.29	0.82
	C1-L4	899.4	1049.4	80.06	49.93	6.00	9.00	0.03	0.53	0.95
	C1-L5	1048.2	1198.2	80.03	49.89	5.97	8.96	0.03	1.06	1.09
	C1-L6	1199.3	1349.3	79.92	50.10	5.99	8.99	0.03	1.20	1.22
C 120×60×6	C2-L1	399.2	549.2	119.84	60.12	6.02	9.03	0.02	-0.39	0.38
	C2-L2	499.5	649.5	120.02	59.88	6.02	9.03	0.02	0.25	0.45
	C2-L3	600.1	750.1	120.18	59.93	6.01	9.02	0.02	-0.44	0.52
	C2-L4	698.3	848.3	118.99	60.75	5.99	8.99	0.02	0.15	0.59
	C2-L5	799.0	949.0	120.46	59.59	5.97	8.96	0.02	-0.28	0.66
	C2-L6	898.7	1048.7	120.08	60.08	6.00	9.00	0.02	0.17	0.73

7.2.2 Material tests

The detailed test setup and procedures for the material tensile flat and corner coupon tests are described in Chapter 3. The obtained (flat and corner) material stress–strain curves for the two adopted press-braked stainless steel channel sections are shown in Figures 3.3(c) and 3.3(d), respectively, while a summary of the key obtained material properties is given in Table 3.1.

7.2.3 Initial global and local geometric imperfection measurements

The behaviour and resistances of thin-walled metallic structural members are sensitive to the initial geometric imperfections, and therefore the initial global and local geometric imperfections of the press-braked stainless steel channel section column specimens were carefully measured. The setup for initial global geometric imperfection measurements is shown in Figure 7.1, where a column specimen is mounted onto the flat work bench of a CNC router, and two LVDTs, pointing to the web of the column specimen, are moved along two longitudinal lines to measure the deviations about the minor principal axis (i.e. the axis of flexural buckling). The initial mid-height global geometric imperfection of each column

specimen about the minor principal axis was then taken as the deviation from a linear reference line (i.e. a straight line connecting the data points measured at the two ends) to the data point measured at the mid-height. Table 7.1 reports the measured initial mid-height global geometric imperfection amplitude of each press-braked stainless steel channel section column specimen ω_g . It is worth noting that the measured initial mid-height global geometric imperfection amplitude is taken as positive if the column specimen initially bows towards the flange tips, but negative if the column specimen initially bows towards the web, as indicated in Figure 7.2. Direct measurements of the initial local geometric imperfections on the column specimens were significantly impeded by the existence of initial global geometric imperfections; therefore, their magnitudes ω_0 were taken as those previously measured from the corresponding stub column specimens with the same cross-section sizes in Chapter 3, and are summarised in Table 7.1.

Figure 7.1: Test setup for initial global geometric imperfection measurements

the distance from the rotation centre of the knife-edge device to the end section of the specimen is equal to 75 mm, resulting in the effective length of each column specimen L_e equal to the measured specimen length L plus an additional length of 150 mm, and the corresponding member non-dimensional slenderness about the minor principal axis (i.e. the axis of buckling) is calculated from Equation (7.1), where I_z is the second moment of area about the minor principal axis, as reported in Table 7.1.

$$\bar{\lambda}_z = \sqrt{\frac{A\sigma_{0.2}L_e^2}{\pi^2 EI_z}} \quad (7.1)$$

Figure 7.3: Pin-ended column test setup

Figure 7.4: Sign convention for initial loading eccentricity e_{0e}

An LVDT, horizontally pointing to the centreline of the web at mid-height, was adopted to measure the lateral deflection of the column specimen along the buckling direction, and two strain gauges, attached to the outer faces of the web and flange at the mid-height of the specimen and symmetrically in relation to the cross-section minor principal axis, were used to record the longitudinal strains at the two positions. The readings from the LVDT and strain gauges were used to derive the initial loading eccentricity at the specimen end, based on Equation (7.2) (Rasmussen and Hancock, 1995; Zhao et al., 2016c; Chen and Young, 2018; Buchanan et al., 2018), where e_{0e} is the initial loading eccentricity from the cross-section elastic neutral axis at the specimen end, d is the distance between the two strain gauges, N is the applied compression load, Δ is the lateral deflection at the mid-height of the specimen measured by the LVDT and $\varepsilon_{max} - \varepsilon_{min}$ is the difference of the longitudinal strains measured by the two strain gauges. Note that Equation (7.2) was derived by assuming that the structural behaviour was close to linear elastic, and it was suggested that no more than 15% of the expected failure load be used in the calculation of e_{0e} (Zhao et al., 2016c; Buchanan et al., 2018). The initial loading eccentricity is taken as positive if the resulting initial end moments lead the specimen to bow towards the flange tips, but negative if the resulting initial end moments lead the specimen to bow towards the web – see Figure 7.4; this is consistent with

the sign convention for initial global geometric imperfections. It is worth noting that if the absolute value of the sum of the measured initial global geometric imperfection at the specimen mid-height and the calculated initial loading eccentricity at the specimen end $/\omega_g + e_{0e}/$ was larger than $L_e/1000$, the position of the column specimen was then carefully re-adjusted until the attainment of $/\omega_g + e_{0e}/ < L_e/1000$ (Rasmussen and Hancock, 1995; Buchanan et al., 2018). Table 7.2 reports the overall global geometric imperfection amplitude, defined as the sum of the initial mid-height global geometric imperfection amplitude and initial end loading eccentricity ($\omega_g + e_{0e}$), together with the ratio of its absolute value to the effective member length $/\omega_g + e_{0e}/L_e$, for each press-braked stainless steel channel section column specimen.

$$e_{0e} = \frac{EI_z(\varepsilon_{max} - \varepsilon_{min})}{Nd} - \Delta - \omega_g \quad (7.2)$$

All the press-braked stainless steel channel section column specimens underwent noticeable global deformations upon testing, and failed by visible member flexural buckling about the minor principal axis. Two orientations associated with minor-axis flexural buckling were observed for the failure specimens, namely ‘C’ orientation (indicating that the failure specimens buckled towards the webs) and ‘reverse C’ orientation (indicating that the failure specimens buckled towards the flange tips). With regard to channel section column specimens with negative overall global geometric imperfection amplitudes, the induced second-order bending moments led to compressive stresses at the flange tips; for these cases, the failure modes displayed minor-axis flexural buckling in the ‘C’ orientation, with a typical failure specimen C2-L3 shown in Figure 7.5. For channel section column specimens with positive overall global geometric imperfection amplitudes, the induced second-order bending moments resulted in tensile stresses at the flange tips and the corresponding failure modes showed minor-axis flexural buckling in the ‘reverse C’ orientation; a typical failure specimen C1-L4 is presented in Figure 7.6. The load–mid-height lateral deflection curves measured for the C 80×50×6 and C 120×60×6 column specimens are displayed in Figures 7.7(a) and 7.7(b), respectively, while a summary of the key obtained test results, including the failure load $N_{u,test}$, the mid-height lateral deflection at the failure load Δ_u and the buckling orientation, are

reported in Table 7.2; the mid-height lateral deflections are taken as negative for channel section column specimens failing by minor-axis flexural buckling in the ‘C’ orientation, but are positive if the failure specimens buckle in the ‘reverse C’ orientation, which are consistent with the sign convention for initial global geometric imperfections and initial end loading eccentricities.

Figure 7.5: Experimental and numerical failure modes for specimen C2-L3 (with minor-axis flexural buckling in the ‘C’ orientation)

Figure 7.6: Experimental and numerical failure modes for specimen C1-L4 (with minor-axis flexural buckling in the ‘reverse C’ orientation)

(a) C 80×50×6

(b) C 120×60×6

Figure 7.7: Experimental load–mid-height lateral deflection curves of pin-ended press-braked stainless steel channel section column specimens

Table 7.2: Summary of pin-ended press-braked stainless steel channel section column test results

Cross-section	Specimen ID	e_{0e} (mm)	$\omega_g + e_{0e}$ (mm)	$ \omega_g + e_{0e} /L_e$	Orientation	Δ_u (mm)	$N_{u,test}$ (kN)	$N_{u,FE}/N_{u,test}$
C 80×50×6	C1-L1	0.20	-0.20	1/2999	C	-2.00	279.5	0.95
	C1-L2	0.30	-0.19	1/4048	C	-1.86	274.7	0.94
	C1-L3	0.30	0.01	1/62514	C	-0.41	269.5	0.99
	C1-L4	-0.15	0.38	1/2770	reverse C	4.22	261.5	0.99
	C1-L5	-0.25	0.81	1/1479	reverse C	6.36	248.4	1.04
	C1-L6	-0.16	1.04	1/1297	reverse C	2.23	228.1	1.00
C 120×60×6	C2-L1	0.67	0.28	1/1959	C	0.71	461.1	1.00
	C2-L2	-0.20	0.05	1/12990	C	-1.42	447.7	1.02
	C2-L3	-0.30	-0.74	1/1008	C	-1.97	392.5	0.97
	C2-L4	-0.60	-0.45	1/1896	C	-1.74	400.5	0.99
	C2-L5	0.30	0.02	1/49974	C	-0.19	407.5	1.00
	C2-L6	0.11	0.28	1/3799	reverse C	4.37	409.3	1.00
							Mean	0.99
							COV	0.03

It is worth noting that the mid-height lateral deflections of four press-braked stainless steel channel section column specimens C1-L3, C2-L1, C2-L2 and C2-L5 changed the direction during testing. For each of these four cases, the signs of the measured initial global geometric imperfection and initial end loading eccentricity are opposite (i.e. the initial global geometric imperfection and the initial end loading eccentricity lead the specimen to deflect in different directions), and the magnitudes of the initial global geometric imperfection and the initial end loading eccentricity are comparable, with $|e_{0e}|$ greater than $|\omega_g|$. Therefore, the end moments resulted from the initial end loading eccentricity initially governed the structural behaviour and deflection of the column specimen during the early stage of loading; as the compression load increased, the mid-height bending moment resulted from the initial global geometric imperfection became dominant and forced the specimen to deflect in the reverse direction (Buchanan et al., 2018).

7.3 Numerical modelling programme

7.3.1 General

In conjunction with the experimental programme, a numerical modelling programme was conducted by means of the nonlinear finite element software ABAQUS (ABAQUS, 2012). FE models were firstly developed and validated against the experimental results, and then adopted to perform parametric studies to generate further numerical data on press-braked stainless steel channel section columns failing by minor-axis flexural buckling over a wide range of cross-section dimensions and member effective lengths.

7.3.2 Development of FE models

Each press-braked stainless steel channel section column was modelled based on the measured cross-section dimensions and member effective length and using the four-node shell element S4R, which has been widely used in previous FE modelling of thin-walled steel channel section structural members (Dobrić et al., 2017, 2020; Liang et al., 2019c, 2019d,

2020; Zhang et al., 2019b, 2020a, 2020b, 2020e; Dissanayake et al., 2020; Wang et al., 2020a, 2020b; Liang et al., 2019c, 2019d, 2020; Zhao and Gardner, 2018; Becque and Rasmussen, 2009b). Regarding the element width along the centreline of the cross-section, a uniform element width equal to the wall thickness t was assigned to the flat portions, while each corner of the cross-section was uniformly discretised into four elements to ensure an accurate representation of the curved geometry. The element length along the longitudinal direction of the channel section column FE model was equal to the wall thickness t . The material true stress–plastic strain curves, as converted from the measured engineering stress–strain curves of the respective flat and corner coupons, were assigned to the corresponding flat and corner portions of the press-braked stainless steel channel section column FE models. Residual stresses were not incorporated into the press-braked stainless steel channel section column FE models, as discussed in Chapter 3. For ease of application of boundary conditions, each end section of the channel section column FE model was coupled to an eccentric reference point, which is located near the flange tip side if the measured initial end loading eccentricity e_{0e} (see Table 7.2) is negative, but at the other side if e_{0e} is positive; moreover, one reference point was fully restrained except for rotation about the minor principal axis (i.e. the axis of buckling), while the other reference point was allowed for translation in the longitudinal direction and rotation about the minor principal axis, to replicate the same pin-ended boundary conditions as those adopted in the tests. In terms of the incorporation of initial global and local geometric imperfections into the channel section column FE models, the lowest elastic global (minor-axis flexural) and local buckling mode shapes (Chen and Young, 2018; Dissanayake et al., 2020; Wang et al., 2020b), obtained from an elastic eigenvalue buckling analysis, were taken as the initial global and local geometric imperfection distribution patterns, respectively; moreover, the global buckling mode shape of each channel section column FE model was oriented such that it was consistent with the measured initial global geometric imperfection shape of the corresponding specimen. The measured initial global and local geometric imperfection magnitudes $|\omega_g|$ and ω_0 were then adopted to factor the respective obtained initial geometric imperfection distribution patterns. Upon development of the press-braked stainless steel channel section column FE models, the modified Riks method, commonly adopted for solving static numerical problems with material and geometrical nonlinearities (Chen and Young, 2018; Dissanayake et al., 2020;

Wang et al., 2020b), was employed in the present study for nonlinear analyses of the developed column FE models.

7.3.3 Validation of FE models

The numerical failure loads, load–mid-height lateral deflection curves and failure modes, as derived from the Riks analysis, were carefully compared with those obtained from the tests, allowing the accuracy of the developed column FE models to be assessed. The numerical to test failure load ratios $N_{u,FE}/N_{u,test}$ for the twelve press-braked stainless steel channel section column specimens are reported in Table 7.2, with the mean $N_{u,FE}/N_{u,test}$ ratio equal to 0.99 and the corresponding COV equal to 0.03, indicating excellent agreement between the test and numerical failure loads. Moreover, the developed column FE models are also shown to be capable of simulating the experimental load–mid-height lateral deflection curves, as evident in Figures 7.8 and 7.9, where the experimental and numerical load–mid-height lateral deflection curves for two typical press-braked stainless steel channel section column specimens C1-L4 and C2-L3 are compared. Good agreement on failure modes was also observed, as shown in Figures 7.5 and 7.6, where the test failure modes for the aforementioned specimens C2-L3 and C1-L4 are compared with their numerical counterparts, respectively. Overall, the developed FE models can accurately simulate the test responses of the pin-ended press-braked stainless steel channel section column specimens, and are thus deemed to be validated.

Figure 7.8: Experimental and numerical load–mid-height lateral deflection curves for specimen C1-L4

Figure 7.9: Experimental and numerical load–mid-height lateral deflection curves for specimen C2-L3

The FE models were also developed in the following way: the end sections were coupled to concentric reference points and the effect of the initial end loading eccentricities was considered through amplifying the initial mid-height global geometric imperfections; note that this modelling method is consistent with that used for deriving the design column buckling curves in the European codes (CEN, 2014a, 2015). Two global geometric imperfection magnitudes, including $L_e/1000$ and $L_e/1500$, were selected to factor the initial global geometric imperfection distribution patterns, while two local geometric imperfection values, including $t/10$ and $t/100$, were used to scale the initial local geometric imperfection patterns. Four combinations of global and local geometric imperfection magnitudes, including $L_e/1000$ and $t/10$, $L_e/1500$ and $t/10$, $L_e/1000$ and $t/100$ as well as $L_e/1500$ and $t/100$, were examined. Table 7.3 reports the numerical to test failure load ratios for the twelve press-braked stainless steel channel section column specimens, with the results revealing that (i) the failure loads were generally insensitive to the local geometric imperfection magnitudes since the studied non-slender channel section columns are not vulnerable to local buckling failure and (ii) the combination of the local geometric imperfection magnitude of $t/100$ (or $t/10$) and the global geometric imperfection magnitude of $L_e/1000$ results in accurate predictions of the test failure loads.

Table 7.3: Comparisons of numerical failure loads with test failure loads for varying initial geometric imperfection magnitude combinations

Cross-section	Specimen ID	$N_{u,FE}/N_{u,test}$			
		$L_e/1000+t/10$	$L_e/1500+t/10$	$L_e/1000+t/100$	$L_e/1500+t/100$
C 80×50×6	C1-L1	1.10	1.15	1.10	1.16
	C1-L2	1.08	1.15	1.08	1.16
	C1-L3	1.08	1.17	1.08	1.15
	C1-L4	1.07	1.16	1.08	1.16
	C1-L5	1.04	1.18	1.05	1.17
	C1-L6	1.07	1.02	1.08	1.02
C 120×60×6	C2-L1	0.93	0.93	0.94	0.94
	C2-L2	0.93	0.93	0.94	0.94
	C2-L3	1.07	1.11	1.06	1.13
	C2-L4	1.03	1.14	1.03	1.10
	C2-L5	0.96	0.99	0.95	0.99
	C2-L6	0.90	0.94	0.91	0.94
Mean		1.02	1.07	1.02	1.07
COV		0.07	0.09	0.07	0.09

7.3.4 Parametric studies

On the basis of the validated FE models, parametric studies were performed to generate further numerical data on press-braked stainless steel channel section columns over a wide range of cross-section dimensions and member effective lengths. In the parametric studies, the second modelling method was used, i.e. the end sections of each channel section column FE model were coupled to concentric reference points. Given that the combination of the global geometric imperfection magnitude of $L_e/1000$ and local geometric imperfection magnitude of $t/100$ leads to the best agreement between the test and numerical failure loads in the validation study, these two magnitudes were also used for the channel section column FE models in the parametric studies. For each modelled channel section column, the initial global geometric imperfection pattern, bowing towards the web, was assigned, i.e. the resulting second-order bending moment induced compressive stresses at the flange tips and the corresponding flexural buckling failure mode was in the ‘C’ orientation; note that in comparison with flexural buckling in the ‘reverse C’ orientation, flexural buckling in the ‘C’ orientation is more unfavourable and critical, with smaller failure load. The measured flat and corner material stress–strain curves of the channel section C 120×60×6 were used in the parametric studies. With regard to the cross-section dimensions of the modelled channel sections, the outer web widths were fixed to 150 mm, while three outer flange widths equal to 125 mm, 100 mm and 75 mm were selected, leading to three cross-section web-to-flange aspect ratios – 1.2, 1.5 and 2.0 being considered. Moreover, the wall thicknesses were varied between 8 mm and 20 mm, and the inner corner radii were set to be equal to 1.5 times the corresponding wall thicknesses. A total of nine channel sections were finally modelled in the parametric studies, and all the modelled channel sections were categorised as non-slender cross-sections according to EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). For each modelled channel section, eighteen effective column lengths, ranged from 800 mm to 6680 mm, were selected. Table 7.4 summarises the detailed cross-section dimensions and member effective lengths of the modelled channel section columns. Overall, 162 numerical data on press-braked stainless steel channel section columns failing by minor-axis flexural buckling were generated from the parametric studies.

Table 7.4: Cross-section dimensions and effective member lengths selected for parametric studies

H (mm)	B (mm)	t (mm)	r (mm)	L_e (mm)
150	75	12	18	800, 1000, 1200, 1400, 1600, 1800, 2000,
150	75	10	15	2200, 2400, 2600, 2800, 3000, 3200, 3400,
150	75	8	12	3600, 3800, 4000, 4200
150	100	16	24	1100, 1400, 1700, 2000, 2300, 2600, 2900,
150	100	14	21	3200, 3500, 3800, 4100, 4400, 4700, 5000,
150	100	12	18	5300, 5600, 5900, 6200
150	125	20	30	1240, 1560, 1880, 2200, 2520, 2840, 3160,
150	125	17	26	3480, 3800, 4120, 4440, 4760, 5080, 5400,
150	125	14	21	5720, 6040, 6360, 6680

7.4 Assessment of existing design methods

7.4.1 General

In this section, the accuracy of the design rules for press-braked stainless steel channel section columns failing by minor-axis flexural buckling, as given in the European code EN 1993-1-4 (CEN, 2015), American specification SEI/ASCE 8-02 (ASCE, 2002), Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) and design manual SCI P413 (SCI, 2017), was assessed by comparing the test and numerical failure loads N_u with the unfactored design flexural buckling resistances $N_{u,pred}$. Table 7.5 reports the mean test (and numerical) to predicted failure load ratio $N_u/N_{u,pred}$ and the corresponding COV for each design method.

Table 7.5: Comparisons of test and numerical failure loads with predicted flexural buckling resistances

$\bar{\lambda}_z$	No. of tests	No. of FE data	$N_u/N_{u,EC3}$		$N_u/N_{u,ASCE}$		$N_u/N_{u,AS/NZS}$		$N_u/N_{u,SCI}$	
			Mean	COV	Mean	COV	Mean	COV	Mean	COV
0.38–1.00	10	44	0.92	0.11	0.95	0.10	1.18	0.12	1.14	0.12
1.00–2.81	2	118	1.14	0.07	0.94	0.07	1.26	0.09	1.33	0.08
0.38–2.81	12	162	1.07	0.12	0.94	0.08	1.23	0.11	1.27	0.11

7.4.2 EN 1993-1-4

The European code EN 1993-1-4 (CEN, 2015) provides supplementary design rules for stainless steel structures in addition to the general design rules for mild steel structures set out in EN 1993-1-1 (CEN, 2014a). With regard to the design of column members failing by global buckling (e.g., flexural, torsional and flexural-torsional buckling), the concept of buckling curves, as developed based on the Perry-Robertson buckling formula, is adopted. Four buckling curves are defined in EN 1993-1-4 (CEN, 2015) and reflect the different effects of initial geometric imperfections and residual stresses on the column buckling resistances. The EC3 design resistance of a column failing by global buckling $N_{u,EC3}$ is calculated from Equation (7.3), where χ is the reduction factor and determined from the EC3 design buckling curves, as given by Equation (7.4), where ϕ is a buckling parameter, as calculated from Equation (7.5), where α_{EC3} is the imperfection factor and $\bar{\lambda}_0$ is the limiting slenderness; note that the values of α_{EC3} and $\bar{\lambda}_0$ are respectively equal to 0.49 and 0.4 for cold-formed stainless steel channel section columns failing by flexural buckling about the minor principal axis.

$$N_{u,EC3} = \chi A \sigma_{0.2} \quad (7.3)$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_z^2}} \leq 1 \quad (7.4)$$

$$\phi = 0.5 \left[1 + \alpha_{EC3} (\bar{\lambda}_z - \bar{\lambda}_0) + \bar{\lambda}_z^2 \right] \quad (7.5)$$

The accuracy of the EC3 design buckling curve was assessed through comparisons of the predicted flexural buckling resistances with the test and numerical failure loads. The mean ratio of the test and numerical failure loads to the EC3 design flexural buckling resistances $N_u/N_{u,EC3}$ is equal to 1.07, with the corresponding COV of 0.12, as reported in Table 7.5, revealing that the EC3 design buckling curve yields a good level of design accuracy on average. However, it should be noted that the mean ratio of $N_u/N_{u,EC3}$ for press-braked stainless steel channel section columns with member non-dimensional slendernesses less than 1.0 is equal to 0.92, indicating unsafe flexural buckling resistance predictions. Graphical

assessment of the EC3 design buckling curve is shown in Figure 7.10, where the test and numerical failure loads, normalised by the cross-section yield loads $A\sigma_{0.2}$ and plotted against the member non-dimensional slendernesses $\bar{\lambda}_z$, are compared with the EC3 design buckling curve. It can be observed that the EC3 design buckling curve results in a large number of unsafe flexural buckling resistance predictions for press-braked stainless steel channel section columns with member non-dimensional slendernesses less than around 1.0. Therefore, it may be concluded that the existing EC3 design buckling curve is not suitable for press-braked stainless steel channel section columns failing by flexural buckling about the minor principal axis.

Figure 7.10: Comparisons of test and numerical failure loads with design buckling curves from EN 1993-1-4, AS/NZS 4673 and SCI P413

7.4.3 SEI/ASCE 8-02

The American specification SEI/ASCE 8-02 (ASCE, 2002) uses the allowable stress design method to determine the resistances of cold-formed stainless steel columns. The ASCE design resistance of a column prone to global buckling $N_{u,ASCE}$ is given by Equation (7.6), where F_n is the design allowable stress and calculated with the consideration of all the

potential flexural, torsional and flexural-torsional buckling failure modes. With regard to channel section columns that are not prone to torsional and flexural-torsional buckling (as are the cases herein), the design allowable stress shall be taken as the corresponding design stress for flexural buckling about the minor principal axis, as calculated from Equation (7.7), where E_t is the tangent modulus of the material stress–strain curve at the design allowable stress F_n ; note that cumbersome iterations are generally involved in the determination of E_t and F_n .

$$N_{u,ASCE} = AF_n \quad (7.6)$$

$$F_n = \frac{\pi^2 E_t I_z}{AL_e^2} \quad (7.7)$$

The design flexural buckling resistances of press-braked stainless steel channel section columns, as predicted from SEI/ASCE 8-02 (ASCE, 2002), were assessed through comparisons against the test and numerical failure loads. Quantitative and graphical assessments of the ASCE design flexural buckling resistances are respectively shown in Table 7.5, where the mean ratio of the test and numerical failure loads to the ASCE flexural buckling resistance predictions $N_u/N_{u,ASCE}$ is equal to 0.94 and the corresponding COV is equal to 0.08, and Figure 7.11, where the test and numerical failure loads, normalised by the ASCE flexural buckling resistance predictions, are plotted against the member non-dimensional slendernesses $\bar{\lambda}_z$. The results of the quantitative and graphical assessments indicated that the allowable stress design method set out in SEI/ASCE 8-02 (ASCE, 2002) overpredicts the flexural buckling resistances of press-braked stainless steel channel section columns over the whole considered range of member non-dimensional slendernesses up to 3.0.

Figure 7.11: Comparisons of test and numerical failure loads with ASCE flexural buckling resistance predictions

7.4.4 AS/NZS 4673

The Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) employs the same allowable stress design method as that set out in SEI/ASCE 8-02 (ASCE, 2002) for cold-formed stainless steel columns, but also provides an alternative explicit approach for those prone to flexural buckling only. Similarly to the EC3 design buckling curves, the AS/NZS explicit approach was also established based on the Perry-Robertson buckling formula. The AS/NZS design flexural buckling resistance $N_{u,AS/NZS}$ is calculated from Equation (7.8), where F_a is the design flexural buckling stress and determined from Equation (7.9), where $\phi_{AS/NZS}$ is a buckling parameter and given by Equation (7.10), where η is the imperfection factor and given by Equation (7.11), where the parameters $\alpha_{AS/NZS}$, $\beta_{AS/NZS}$, λ_0 and λ_1 are dependent on the stainless steel grade, and respectively taken as 1.59, 0.28, 0.55 and 0.2 for austenitic stainless steel.

$$N_{u,AS/NZS} = AF_a \quad (7.8)$$

$$F_a = \frac{\sigma_{0.2}}{\phi_{AS/NZS} + \sqrt{\phi_{AS/NZS}^2 - \bar{\lambda}_z^2}} \leq \sigma_{0.2} \quad (7.9)$$

$$\phi_{AS/NZS} = 0.5 \left[1 + \eta + \bar{\lambda}^2 \right] \quad (7.10)$$

$$\eta = \alpha_{AS/NZS} \left[\left(\bar{\lambda}_z - \lambda_1 \right)^{\beta_{AS/NZS}} - \lambda_0 \right] \quad (7.11)$$

Assessment of the accuracy of the AS/NZS explicit approach for the design of press-braked stainless steel channel section columns failing by minor-axis flexural buckling was conducted by comparing the predicted resistances with the test and numerical failure loads. The mean test and numerical to predicted failure load ratio $N_u/N_{u,AS/NZS}$, as reported in Table 7.5, is equal to 1.23, with the corresponding COV of 0.11. Comparisons between the AS/NZS design buckling curve (as defined by Equations (7.8)–(7.11)) and the test and numerical failure loads are graphically shown in Figure 7.10. Overall, AS/NZS 4673 (AS/NZS, 2001) was found to result in more conservative predictions of flexural buckling resistance for press-braked stainless steel channel section columns than EN 1993-1-4 (CEN, 2015) and SEI/ASCE 8-02 (ASCE, 2002), but the number of unsafe flexural buckling resistance predictions has been significantly reduced.

7.4.5 SCI P413 (SCI)

The design manual for structural stainless steel SCI P413 (SCI, 2017) is issued by the Steel Construction Institute. The concept of buckling curves is also used for the determination of the column buckling resistances. With regard to cold-formed stainless steel channel section columns failing by minor-axis flexural buckling, the SCI design buckling curve is defined by Equations (7.3)–(7.5), but with a larger imperfection factor α_{EC3} equal to 0.76 and a smaller limiting slenderness $\bar{\lambda}_0$ equal to 0.2. The SCI design buckling curve, as shown in Figure 7.12, exhibits a good representation of the test and numerical data points over the whole considered range of member non-dimensional slendernesses up to 3.0. The mean test and numerical to predicted failure load ratio $N_u/N_{u,SCI}$, as reported in Table 7.5, is equal to 1.27, with the

corresponding COV of 0.11. In comparison with the EC3 design buckling curve, the SCI design buckling curve leads to significantly reduced number of unsafe flexural buckling resistance predictions for those relatively short and intermediate columns with member non-dimensional slendernesses $\bar{\lambda}_z$ less than 1.0, as evident in Figure 7.10 and Table 7.5. It is worth noting that the imperfection factor $\alpha_{EC3}=0.76$ and limiting slenderness $\bar{\lambda}_0=0.2$, as recommended in SCI P413 (SCI, 2017) for cold-formed stainless steel channel section columns failing by flexural buckling about the minor principal axis, have been included in the next edition of EN 1993-1-4 (CEN, 2015).

7.5 Concluding remarks

Laboratory testing and numerical modelling have been conducted to study the structural behaviour and resistances of pin-ended press-braked stainless steel channel section columns failing by flexural buckling about the minor principal axis. The experimental programme included initial global and local geometric imperfection measurements and twelve pin-ended column tests. The test setup and procedures, together with the key obtained test results, including the failure loads, the mid-height lateral deflections at the failure loads, the load–mid-height lateral deflection curves and the failure modes with flexural buckling in both the ‘C’ and ‘reverse C’ orientations, were fully reported. Following the experimental programme, a numerical modelling programme was conducted; FE models were firstly developed and validated against the test results, and then adopted to perform parametric studies to generate further numerical data on press-braked stainless steel channel section columns over a wide range of cross-section dimensions and member effective lengths. The obtained test and numerical data were adopted to assess the accuracy of the codified design rules, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), and the design method, as given in SCI P413 (SCI, 2017), for press-braked stainless steel channel section columns failing by flexural buckling about the minor principal axis. The results of the assessment revealed that (i) the design buckling curve specified in EN 1993-1-4 (CEN, 2015) overpredicts the flexural buckling resistances for press-braked stainless steel channel section columns with member non-dimensional slendernesses less than around 1.0,

(ii) the allowable stress design method given in SEI/ASCE 8-02 (ASCE, 2002) leads to unsafe predictions of flexural buckling resistance for press-braked stainless steel channel section columns over the whole considered range of member non-dimensional slendernesses up to 3.0, and (iii) the explicit approach set out in AS/NZS 4673 (AS/NZS, 2001) and the design buckling curve recommended in SCI P413 (SCI, 2017) yield more conservative flexural buckling resistance predictions than the methods specified in EN 1993-1-4 (CEN, 2015) and SEI/ASCE 8-02 (ASCE, 2002), but with significantly reduced number of unsafe predictions.

CHAPTER 8

GLOBAL BUCKLING BEHAVIOUR OF CHANNEL SECTION BEAM-COLUMNS

8.1 Introduction

The local buckling behaviour and resistances of press-braked stainless steel channel sections under isolated loading (pure compression and pure bending) and combined loading (compression plus bending about the major axis or minor axis) have been comprehensively studied in Chapters 3–5, respectively, while investigations into the flexural buckling behaviour of press-braked stainless steel channel section columns were reported in Chapter 6. In the present chapter, a testing and numerical modelling programme is conducted to investigate the global buckling behaviour and resistances of press-braked stainless steel channel section beam-columns subjected to compression and bending moment about the minor principal axis. The testing programme is firstly conducted and includes initial geometric imperfection measurements and ten press-braked stainless steel channel section beam-column tests. A parallel numerical modelling programme is then performed, including

a validation study, where finite element models were developed and validated against the test results, and a parametric study, where the validated finite element models are used to generate further numerical data over a wide range of cross-section dimensions, member effective lengths and loading combinations. The obtained test and numerical data are finally used to assess the accuracy and consistency of the codified design rules, as given in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), for press-braked stainless steel channel section beam-columns subject to combined compression load and bending moment about the minor principal axis. The findings of this research have been reported by Zhang et al. (2020g).

8.2 Testing programme

8.2.1 Test specimens

Given that no tests on press-braked stainless steel channel section beam-columns have been reported to date, a testing programme was firstly conducted to obtain a test data bank. Two plain channel sections C 80×50×6 and C 120×60×6, press-braked from grade EN 1.4420 austenitic stainless steel sheets, were adopted in the testing programme. Both of the two adopted press-braked stainless steel channel sections are categorised as Class 3 based on the slenderness limits given in EN 1993-1-4 (CEN, 2015) and also fall in the category of non-slender cross-sections specified in SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). For each channel section, five beam-column specimens with the same nominal member length were prepared and planned to be tested under varying combinations of compression load and minor-axis bending moment. The specimen ID includes a number ‘1’ or ‘2’ (‘1’ signifying the C 80×50×6 specimen series and ‘2’ signifying the C 120×60×6 specimen series) and a letter from ‘A’ to ‘E’ (designating the specimens subjected to different loading combinations within each specimen series). The measured geometric dimensions of each press-braked stainless steel channel section beam-column specimen are reported in Table 8.1.

Table 8.1: Measured geometric dimensions and initial geometric imperfection magnitudes of press-braked stainless steel channel section beam-column specimens

Cross-section	Specimen ID	H (mm)	B (mm)	t (mm)	r (mm)	L (mm)	L_e (mm)	$\bar{\lambda}_z$	ω_0 (mm)	ω_g (mm)
C 80×50×6	1A	80.08	49.97	5.96	8.94	799.8	939.8	0.85	0.03	0.66
	1B	80.11	49.89	5.95	8.93	800.2	940.2	0.85	0.03	0.32
	1C	79.94	50.06	5.96	8.94	800.0	940.0	0.85	0.03	0.50
	1D	80.06	49.93	6.00	9.00	799.7	939.7	0.85	0.03	0.61*
	1E	79.92	50.10	5.99	8.99	799.8	939.8	0.85	0.03	0.64*
C 120×60×6	2A	119.84	60.12	6.02	9.03	849.9	989.9	0.69	0.02	0.37
	2B	120.02	59.88	6.02	9.03	850.1	990.1	0.69	0.02	0.43
	2C	120.18	59.93	6.01	9.02	850.0	990.0	0.69	0.02	0.62
	2D	118.99	60.75	5.99	8.99	850.0	990.0	0.69	0.02	0.63
	2E	120.08	60.08	6.00	9.00	850.1	990.1	0.69	0.02	0.69

* The two specimens initially bowed towards web, while all the other specimens initially bowed towards flange tips.

8.2.2 Material tests

The detailed test setup and procedures for the material tensile flat and corner coupon tests are described in Chapter 3. The obtained (flat and corner) material stress–strain curves for the two adopted press-braked stainless steel channel sections are shown in Figures 3.3(c) and 3.3(d), respectively, while a summary of the key obtained material properties is given in Table 3.1.

8.2.3 Initial global and local geometric imperfection measurements

The behaviour and resistances of thin-walled metallic structural members are sensitive to the initial geometric imperfections, and therefore the initial global and local geometric imperfections of the press-braked stainless steel channel section beam-column specimens were carefully measured. The test setup for initial global geometric imperfection measurements is shown in Figure 8.1, where a beam-column specimen is mounted onto the flat work bench of a CNC router and an LVDT, pointing to the web of the specimen, is moved along its centreline. The initial mid-height global geometric imperfection of each beam-column specimen about the minor principal axis was then taken as the deviation from a linear reference line (i.e., a straight line connecting the data points measured at the two ends)

to the data point measured at the mid-height. Table 8.1 reports the measured initial mid-height global geometric imperfection amplitude about the minor principal axis ω_g for each press-braked stainless steel channel section beam-column specimen. It is worth noting that the beam-column specimens 1D and 1E initially bowed towards flange tips, while all the other beam-column specimens initially bowed towards webs; the two types of initial global geometric imperfection pattern are indicated in Figures 8.2(a) and 8.2(b), respectively. Direct measurements of the initial local geometric imperfections on the beam-column specimen were significantly impeded by the existence of the initial global geometric imperfections (Zhang et al., 2020e). Therefore, their magnitudes ω_0 were taken as those previously measured from the corresponding stub column specimens with the same cross-section sizes in Chapter 3, and are summarised in Table 8.1.

Figure 8.1: Test setup for initial global geometric imperfection measurements

(a) Initial global geometric imperfection pattern for specimen initially bowing towards flange tips

(b) Initial global geometric imperfection pattern for specimen initially bowing towards web

Figure 8.2: Illustration of initial global geometric imperfection pattern

8.2.4 Beam-column (eccentric compression) tests

Eccentric compression tests on the ten press-braked stainless steel channel section beam-column specimens were conducted to investigate their global buckling behaviour and resistances under combined compression and minor-axis bending. The eccentric compression loads were applied to the specimens with varying initial eccentricities to the cross-section elastic neutral axis, which led to a wide range of compression load-to-bending moment ratios being considered. It is also worth noting that when the eccentric compression loads were located between the cross-section elastic principal axes and flange tips, the resulting end moments induced the maximum compressive stresses at the flange tips; for the five C

80×50×6 beam-column specimens, the eccentric compression loads were applied in this way. When the eccentric compression loads were located between the cross-section elastic principal axes and webs, the resulting end moments induced the maximum compressive stresses in the webs; for the five C 120×60×6 beam-column specimens, the eccentric compression loads were applied in this way. An INSTRON 5000 kN hydraulic testing machine was used for all the eccentric compression tests, with the loading rate equal to 0.2 mm/min. A photograph and a schematic diagram of the eccentric compression test setup are shown in Figures 8.3(a) and 8.3(b), respectively. Each end of the testing machine was equipped with a knife-edge device, including a pit plate with a semi-circular groove and a wedge plate with a knife-edge wedge, to offer pin-ended boundary conditions to the specimen ends about the minor principal axis (i.e. the axis of buckling). Prior to testing, each channel section beam-column specimen was welded to 15 mm thick steel plates at both ends, which helped to prevent the specimen ends from twisting and warping under loading. The channel section beam-column specimen welded with end plates was then placed between the top and bottom wedge plates and their relative position was carefully adjusted, to achieve the pre-specified loading eccentricity and also ensure that the member longitudinal axis was perpendicular to the wedge plates and the cross-section minor principal axis was parallel to the knife-edges. Note that the distance from the rotation centre of the knife-edge device to the end section of the specimen is equal to 70 mm, resulting in the effective length of each press-braked stainless steel channel section beam-column specimen about the minor principal axis L_e equal to the measured specimen length L plus an additional length of 140 mm, and the corresponding member non-dimensional slenderness about the minor principal axis $\bar{\lambda}_z$ is determined by Equation (8.1), where $N_{cr} = \pi^2 EI_z / L_e^2$ is the Euler buckling load, as reported in Table 8.1.

$$\bar{\lambda}_z = \sqrt{\frac{A\sigma_{0.2}}{N_{cr}}} \quad (8.1)$$

Figure 8.3: Beam-column test setup

With regard to the instrumentation employed for the eccentric compression tests, two pairs of strain gauges (SG1 and SG2 as well as SG3 and SG4 – see Figure 8.3), adhered to the outer faces of the web and flange tips of the beam-column specimen at mid-height, were used to record the maximum and minimum longitudinal strains, and an LVDT, perpendicularly pointing to the centreline of the web at the specimen mid-height, was adopted to measure the lateral deflections about the minor principal axis. The strain gauge and LVDT readings were used together to determine the actual initial loading eccentricity from the cross-section elastic neutral axis e_{0e} , based on Equation (8.2) (Rasmussen and Hancock, 1995; Zhao et al., 2016c; Chen and Young, 2018; Buchanan et al., 2018), where N is the applied eccentric compression load, Δ is the lateral deflection at the mid-height of the specimen measured by the LVDT, and ε_{max} and ε_{min} are the average maximum compressive strain and the average minimum compressive strain (or the average maximum tensile strain, in the cases with large initial loading eccentricities) recorded by the two pairs of strain gauges, respectively. Note that Equation (8.2) was derived by assuming that the structural behaviour was close to linear

elastic and it was thus suggested that no more than 15% of the expected failure load be used in the calculation of e_{0e} (Liang et al., 2019c; Zhang et al, 2020e).

$$e_{0e} = \frac{EI_z(\varepsilon_{max} - \varepsilon_{min})}{Nb} - \Delta - \omega_g \quad (8.2)$$

Table 8.2 reports the key obtained test results, including the initial loading eccentricity from the cross-section elastic neutral axis e_{0e} , the failure load $N_{u,test}$, the mid-height lateral deflection at the failure load Δ_u and the first-order elastic bending moment at the failure load $M_{1st,el,ue}$, the second-order elastic bending moment at the failure load $M_{2nd,el,ue}$ and the second-order inelastic bending moment at the failure load $M_{2nd,iel,ue}$, which are calculated from Equations (8.3)–(8.5), respectively. Figures 8.4 and 8.5 show the load–mid-height lateral deflection curves measured for the two series of press-braked stainless steel channel section beam-column specimens, respectively. The deformed failure modes for the C 80×50×6 beam-column specimens, where the eccentric compression loads were located between the cross-section elastic principal axes and flange tips, featured noticeable minor-axis flexural buckling together with in-plane bending in the ‘C’ orientation, as shown in Figure 8.6. The deformed failure modes for the C 120×60×6 beam-column specimens, where the eccentric compression loads were located between the cross-section elastic principal axes and webs, exhibited noticeable minor-axis flexural buckling and in-plane bending in the ‘reverse C’ orientation, as shown in Figure 8.7.

$$M_{1st,el,ue} = N_{u,test} (e_{0e} + \omega_g) \quad (8.3)$$

$$M_{2nd,el,ue} = \frac{N_{u,test} (e_{0e} + \omega_g)}{1 - \frac{N_{u,test}}{N_{cr}}} \quad (8.4)$$

$$M_{2nd,iel,ue} = N_{u,test} (e_{0e} + \omega_g + \Delta_u) \quad (8.5)$$

Table 8.2: Summary of press-braked stainless steel channel section beam-column test results

Cross-section	Failure orientation	Specimen ID	e_{0e} (mm)	$N_{u,test}$ (kN)	Δ_u (mm)	$M_{1st,el,ue}$ (kNm)	$M_{2nd,el,ue}$ (kNm)	$M_{2nd,iel,ue}$ (kNm)
C 80×50×6	C	1A	4.4	161.5	4.3	0.6	0.9	1.3
		1B	9.3	118.1	6.5	1.1	1.4	1.8
		1C	14.6	95.4	8.9	1.3	1.7	2.2
		1D	19.4	83.6	9.4	1.7	2.0	2.5
		1E	29.8	67.4	13.4	2.1	2.4	3.0
C 120×60×6	Reverse C	2A	12.9	215.7	11.9	2.9	3.8	5.4
		2B	19.4	183.7	12.7	3.6	4.6	6.0
		2C	23.7	160.1	12.5	3.9	4.7	5.9
		2D	29.0	137.3	14.8	4.1	4.8	6.1
		2E	39.9	106.5	15.1	4.3	4.9	5.9

Figure 8.4: Load–mid-height lateral deflection curves for C 80×50×6 beam-column specimens

Figure 8.5: Load–mid-height lateral deflection curves for C 120×60×6 beam-column specimens

Figure 8.6: Experimental and numerical failure modes for specimen 1B with failure in the ‘C’ orientation

Figure 8.7: Experimental and numerical failure modes for specimen 2A with failure in the ‘reverse C’ orientation

8.3 Numerical modelling programme

8.3.1 General

In conjunction with the testing programme, a numerical modelling programme was conducted by using the nonlinear finite element software ABAQUS (ABAQUS, 2012). Development of press-braked stainless steel channel section beam-column FE models was described in Section 8.3.2, while validation of the developed FE models against the test results was reported in Section 8.3.3. The validated FE models were then used to perform parametric studies to generate further numerical data on press-braked stainless steel channel section beam-columns over a wide range of cross-section dimensions, member effective lengths and loading combinations, as described in Section 8.3.4.

8.3.2 Development of FE models

Each press-braked stainless steel channel section beam-column specimen was modelled based on the measured cross-section dimensions and member effective length, as reported in Table 8.1, with the same modelling assumptions, procedures and techniques as those described in Section 7.3.2 employed herein. For ease of application of boundary conditions, each end section of the channel section beam-column FE model was coupled to an eccentric reference point, with the eccentricity equal to that employed in the tests – see Table 8.3. Proper boundary conditions were afterwards assigned to the reference points; one reference point was fully restrained except for rotation about the minor principal axis, while the other reference point was allowed for translation in the longitudinal direction and rotation about the same minor principal axis, to replicate the pin-ended boundary conditions used in the tests. Initial global and local geometric imperfections were incorporated into each channel section beam-column FE model, with the respective patterns taken as the lowest elastic global (minor-axis flexural) and local buckling mode shapes (Dobrić et al., 2020; Zhang et al., 2020e; Zhu et al., 2019; Chen and Young, 2018; Buchanan et al., 2018), which can be obtained from a prior elastic eigenvalue buckling analysis (ABAQUS, 2012). The measured global imperfection magnitude ω_g and local imperfection magnitude ω_0 were used to factor the respective initial global and local geometric imperfection patterns. Moreover, another four sets of commonly used generalised global and local imperfection magnitude combinations, including $L_e/1000$ and $t/10$, $L_e/1500$ and $t/10$, $L_e/1000$ and $t/100$ and $L_e/1500$ and $t/100$, were also adopted to factor the corresponding initial geometric imperfection patterns to assess the sensitivity of the developed press-braked stainless steel channel section beam-column FE models to imperfection magnitudes.

8.3.3 Validation of FE models

Upon development of the press-braked stainless steel channel section beam-column FE models, nonlinear analyses were conducted using the modified Riks method (ABAQUS, 2012) to derive the numerical failure loads, load–mid-height lateral deflection curves and failure modes. The numerically derived results were then compared with their experimental

observations, allowing the accuracy of the developed FE models to be assessed. The numerical to test failure load ratios $N_{u,FE}/N_{u,test}$ for the ten press-braked stainless steel channel section beam-column specimens are reported in Table 8.3, indicating that good agreement between the test and numerical failure loads is generally obtained for all the five considered (i.e., measured and generalised) imperfection magnitude combinations. Figures 8.6 and 8.7 show comparisons between the test and numerical failure modes for two typical press-braked stainless steel channel section beam-column specimens 1B and 2A with buckling in the ‘C’ and ‘reverse C’ orientations, respectively, also indicating good agreement. Moreover, the experimental load–mid-height lateral deflection curves are well captured by their numerical counterparts, as illustrated in Figures 8.8 and 8.9, where the experimental and numerical load–mid-height lateral deflection curves for the two aforementioned typical specimens 1B and 2A are compared. Overall, the developed FE models can accurately simulate the test responses of the press-braked stainless steel channel section beam-column specimens and are thus deemed to be validated.

Table 8.3: Comparisons of numerical failure loads with test failure loads for varying initial geometric imperfection magnitude combinations

Cross-section	Failure Orientation	Specimen ID	$N_{u,FE}/N_{u,test}$				
			$\omega_g+\omega_0$	$L_e/1000+t/10$	$L_e/1500+t/10$	$L_e/1000+t/100$	$L_e/1500+t/100$
C 80×50×6	C	1A	0.929	0.956	0.935	0.959	0.938
		1B	0.957	0.979	0.968	0.981	0.978
		1C	0.979	0.991	0.983	0.993	0.984
		1D	0.949	0.943	0.947	0.944	0.950
		1E	0.943	0.938	0.943	0.940	0.944
C 120×60×6	Reverse C	2A	0.988	0.971	0.980	0.974	0.981
		2B	0.948	0.935	0.943	0.937	0.945
		2C	0.961	0.953	0.960	0.957	0.963
		2D	0.981	0.974	0.981	0.976	0.984
		2E	1.005	1.000	1.006	1.001	1.007
		Mean	0.964	0.964	0.964	0.966	0.967
		COV	0.024	0.023	0.024	0.023	0.023

Figure 8.8: Experimental and numerical load–mid-height lateral deflection curves for specimen 1B with failure in the ‘C’ orientation

Figure 8.9: Experimental and numerical load–mid-height lateral deflection curves for specimen 2A with failure in the ‘reverse C’ orientation

8.3.4 Parametric studies

The validated FE models were now employed to perform parametric studies to generate further numerical data on press-braked stainless steel channel section beam-columns over a wide range of cross-section dimensions, member effective lengths and loading combinations. In the parametric studies, the adopted modelling procedures and techniques were the same as those described in Section 8.3.2, but with the initial global and local imperfection magnitudes respectively fixed at $L_e/1000$ and $t/100$ and the measured flat and corner material stress–strain curves of the channel section C 120×60×6 used in the material modelling. The cross-section dimensions were carefully selected to ensure that the numerically modelled channel sections covered Class 1–3 cross-sections defined in EN 1993-1-4 (CEN, 2015) and also fell into the category of non-slender cross-sections specified in SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). Specifically, the outer web widths were fixed at 150 mm, while two flange widths equal to 75 mm and 100 mm were selected, leading to two cross-section web-to-flange aspect ratios of 2.0 and 1.5 being considered. The wall thicknesses were varied between 8 mm and 16 mm and the inner corner radii were set to be equal to 1.5 times the corresponding wall thicknesses. For each modelled channel section, six effective member lengths varied between 800 mm and 6000 mm were selected to cover a broad variety of member non-dimensional slendernesses $\bar{\lambda}_z$ ranging from 0.45 to 2.72. Then, for each developed channel section beam-column FE model, a wide range of initial loading eccentricities varied from 0 mm to 1000 mm were selected and the initial loading eccentricities at both sides of the cross-section neutral axis were considered to study the behaviour and resistances of press-braked stainless steel channel section beam-columns subject to a broad range of combinations of compression and minor-axis bending and failing in both the ‘C’ and ‘reverse C’ orientations. Table 8.4 summarises the detailed cross-section dimensions, member effective lengths and initial loading eccentricities of the modelled press-braked stainless steel channel section beam-columns. Overall, 534 numerical data on press-braked stainless steel channel section beam-columns were generated from the parametric studies.

Table 8.4: Cross-section dimensions and effective member lengths selected for parametric studies

H (mm)	B (mm)	t (mm)	r (mm)	Cross-section class*	L_e (mm)	e_{0e} (mm)	Failure orientation
150	75	12	18	1	800, 1500, 2200, 2900, 3600, 4300	5~500	C, reverse C
150	100	16	24	1	1500, 2400, 3300, 4200, 5100, 6000	5~500	C, reverse C
150	75	10	15	2	800, 1500, 2200, 2900, 3600, 4300	5~500	C, reverse C
150	100	14	21	2	1500, 2400, 3300, 4200, 5100, 6000	5~500	C, reverse C
150	75	8	12	3	800, 1500, 2200, 2900, 3600, 4300	5~500	C, reverse C
150	100	12	18	3	1500, 2400, 3300, 4200, 5100, 6000	5~500	C, reverse C

* The cross-section classes are defined according to EN 1993-1-4 (CEN, 2015).

8.4 Assessment of current design standards

8.4.1 General

In this section, the accuracy of the codified design rules for press-braked stainless steel channel section beam-columns subject to combined compression and minor-axis bending, as given in the European code EN 1993-1-4 (CEN, 2015), American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001), was assessed through comparisons of the test and numerical failure loads N_u with the unfactored design resistances $N_{u,pred}$. The quantitative comparison results for press-braked stainless steel channel section beam-columns with failure in the ‘C’ and ‘reverse C’ orientations are reported in Table 8.5, including the mean $N_u/N_{u,pred}$ ratios and the corresponding COVs. Figures 8.10 and 8.11 show the graphical comparison results, where the test and numerical to predicted failure load ratios $N_u/N_{u,pred}$ are plotted against θ . The parameter $\theta = \tan^{-1}[(N_{u,pred}/N_R)/(M_{u,pred}/M_R)]$ (Zhao et al., 2016d) is introduced to reflect the combination of compression and bending, as also graphically defined in Figure 8.12, where N_R and M_R denote the column minor-axis flexural buckling resistance and cross-section minor-axis bending moment resistance, respectively; note that the value of θ changes from 0° to 90° when the applied loading is varied from pure bending to pure compression.

Table 8.5: Comparisons of test and numerical failure loads with predicted resistances from design codes

Failure orientation	Cross-section class*	No. of test data	No. of FE data	$N_u/N_{u,EC3}$		$N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$)	
				Mean	COV	Mean	COV
C	Class 1 and 2	0	155	1.02	0.10	1.07	0.08
	Class 3	5	93	1.21	0.11	1.07	0.07
	Sub-total	5	248	1.10	0.14	1.07	0.08
Reverse C	Class 1 and 2	0	191	0.92	0.07	1.26	0.15
	Class 3	5	95	1.44	0.12	1.28	0.15
	Sub-total	5	286	1.10	0.25	1.27	0.15
Total		10	534	1.10	0.20	1.18	0.15

* The cross-section classes are defined according to EN 1993-1-4 (CEN, 2015).

(a) Failure in the 'C' orientation

(b) Failure in the 'reverse C' orientation

Figure 8.10: Comparisons of test and numerical failure loads with EN 1993-1-4 resistance predictions

(a) Failure in the 'C' orientation

(b) Failure in the 'reverse C' orientation

Figure 8.11: Comparisons of test and numerical failure loads with SEI/ASCE 8-02 (or AS/NZS 4673) resistance predictions; note that the cross-section classes are defined according to EN 1993-1-4

Figure 8.12: Definition of θ

8.4.2 EN 1993-1-4

The European code EN 1993-1-4 (CEN, 2015) provides supplementary design rules for stainless steel structures in addition to the general design rules for carbon steel structures set out in EN 1993-1-1 (CEN, 2014a). Regarding the design of uniform members subject to combined compression and minor-axis bending, the relevant interaction formula is given by Equation (8.6),

$$\frac{N_{u,EC3}}{N_{b,z,Rd}} + k_z \frac{M_{uz,EC3}}{M_{z,Rd}} \leq 1 \quad (8.6)$$

where $N_{u,EC3}$ is the design failure load, $M_{uz,EC3}$ is the design failure moment, which is taken as the product of $N_{u,EC3}$ and the initial loading eccentricity from the cross-section plastic neutral axis e_{0p} for Class 1 or 2 channel sections but the product of $N_{u,EC3}$ and the initial loading eccentricity from the cross-section elastic neutral axis e_{0e} for Class 3 channel sections, $N_{b,z,Rd}$ is the column minor-axis flexural buckling resistance and determined based on the EC3 design buckling curve for stainless steel cold-formed open section columns with the imperfection factor $\alpha_{EC3}=0.49$ and limiting slenderness $\bar{\lambda}_0=0.4$, $M_{z,Rd}$ is the cross-section bending moment resistance about the minor principal axis, which is taken as the plastic moment resistance $M_{pl,z}=W_{pl,z}\sigma_{0.2}$ ($W_{pl,z}$ is the corresponding plastic section modulus about the minor principal axis) for Class 1 or 2 channel sections but the elastic moment capacity $M_{el,z}=W_{el,z}\sigma_{0.2}$ ($W_{el,z}$ is the corresponding elastic section modulus about the minor principal axis) for Class 3 channel sections, and k_z is the interaction factor and given by Equation (8.7).

$$k_z = 1.0 + 2(\bar{\lambda}_z - 0.5) \frac{N_{u,EC3}}{N_{b,z,Rd}} \quad \text{but} \quad 1.2 \leq k_z \leq 1.2 + 2 \frac{N_{u,EC3}}{N_{b,z,Rd}} \quad (8.7)$$

The accuracy and consistency of the EC3 design interaction formula for press-braked stainless steel channel section beam-columns were assessed, based on the test and numerical failure loads. As reported in Table 8.5, the mean test and numerical to EC3 predicted failure

load ratios $N_u/N_{u,EC3}$ are equal to 1.10 for press-braked stainless steel channel section beam-columns failing in both the ‘C’ and ‘reverse C’ orientations, with the corresponding COVs of 0.14 and 0.25, revealing that the EC3 design interaction formula leads to overall accurate but scattered resistance predictions. However, it is worth noting that the overall good level of design accuracy is actually the compromise of unsafe resistance predictions for Class 1 and 2 press-braked stainless steel channel section beam-columns and overly conservative resistance predictions for Class 3 press-braked stainless steel channel section beam-columns. The above discussions are also supported by the graphical assessment results shown in Figure 8.10, where the test and numerical failure loads N_u , normalised by the EC3 resistance predictions $N_{u,EC3}$, are plotted against θ . The scattered resistance predictions arise due to the shortcomings of the EC3 design interaction formula, including (i) the adoption of unsafe buckling curve for determining the column minor-axis flexural buckling resistance $N_{b,z,Rd}$ (Dobrić et al., 2020; Zhang et al., 2020e), (ii) the neglect of strain hardening in the calculation of the cross-section minor-axis bending moment resistance $M_{z,Rd}$ (Zhang et al., 2020a; Zhao and Gardner, 2018) and (iii) the use of inaccurate interaction factor k_z that is not capable of considering the interaction effect.

8.4.3 SEI/ASCE 8-02 and AS/NZS 4673

The American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001) use the same design interaction formula for cold-formed stainless steel beam-columns subject to combined compression and minor-axis bending, as given by Equation (8.8),

$$\frac{N_{u,ASCE}}{N_n} + \frac{C_{mz}M_{uz,ASCE}}{\alpha_{nz}M_{nz}} = 1 \quad \text{or} \quad \frac{N_{u,AS/NZS}}{N_n} + \frac{C_{mz}M_{uz,AS/NZS}}{\alpha_{nz}M_{nz}} = 1 \quad (8.8)$$

where $N_{u,ASCE}$ (or $N_{u,AS/NZS}$) is the design failure load, $M_{uz,ASCE}=N_{u,ASCE}e_{0e}$ (or $M_{uz,AS/NZS}=N_{u,AS/NZS}e_{0e}$) is the design failure moment; note that the ASCE (or AS/NZS) design failure moment is always defined about the cross-section centroid axis, i.e. the cross-section elastic neutral axis, N_n is the nominal axial compression resistance about the minor principal

axis and determined based on the tangent modulus approach specified in Clause 3.4 of SEI/ASCE 8-02 (ASCE, 2002) (or Clause 3.4 of AS/NZS 4673 (AS/NZS, 2001)), M_{nz} is the cross-section bending moment resistance about the minor principal axis and determined based on initiation of yielding for non-slender channel sections with unstiffened compression elements (according to Clause 3.3 of SEI/ASCE 8-02 (ASCE, 2002) or Clause 3.3 of AS/NZS 4673 (AS/NZS, 2001)), resulting in the elastic moment capacity $M_{el,z}$ being adopted, C_{mz} is the equivalent moment factor, with the value equal to unity for beam-columns with equal end moments, and $\alpha_{nz}=1-N_{u,ASCE}/N_{cr}$ (or $\alpha_{nz}=1-N_{u,AS/NZS}/N_{cr}$) is the magnification factor to consider the interaction effect.

The failure loads for press-braked stainless steel channel section beam-columns, as predicted from the ASCE (or AS/NZS) design interaction formula, were assessed through comparisons against the test and numerical failure loads. For press-braked stainless steel channel section beam-columns with failure in the ‘C’ and ‘reverse C’ orientations, the mean $N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$) ratios, as reported in Table 8.5, are equal to 1.07 and 1.27, respectively, with the corresponding COVs of 0.08 and 0.15. Comparisons of the test and numerical failure loads with the resistance predictions from SEI/ASCE 8-02 (ASCE, 2002) (or AS/NZS 4673 (AS/NZS, 2001)) are also shown in Figure 8.11, where the ratios of $N_u/N_{u,ASCE}$ (or $N_u/N_{u,AS/NZS}$) are plotted against θ . On the basis of the quantitative and graphical assessments, the following conclusions can be made. In terms of the design accuracy, SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) yield slightly more accurate resistance predictions than EN 1993-1-4 (CEN, 2015) for press-braked stainless steel channel section beam-columns with failure in the ‘C’ orientation, but result in much more conservative though safe resistance predictions for those ‘reverse C’-orientation buckling cases. In terms of the design consistency, much more consistent resistance predictions were obtained by using SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) in comparison with those calculated from EN 1993-1-4 (CEN, 2015). Similarly to the EC3 design interaction formula of Equation (8.6), the ASCE (or AS/NZS) design interaction formula of Equation (8.8) also suffers from inaccurate predictions of the member minor-axis flexural buckling resistance N_n (Dobrić et al., 2020; Zhang et al., 2020e) and cross-section minor-axis bending moment resistance M_{nz} (Zhang et al., 2020a; Zhao and Gardner, 2018) and inappropriate interaction factor α_{nz} , which

finally lead to unsatisfactory resistance predictions for press-braked stainless steel channel section beam-columns.

8.5 Concluding remarks

The structural behaviour and resistances of press-braked stainless steel channel section beam-columns under combined compression and minor-axis bending have been investigated through a comprehensive testing and numerical modelling programme. The testing programme included initial geometric imperfection measurements and ten eccentric compression tests. The test setup, procedures and results, including the failure loads, mid-height lateral deflections at the failure loads, load–mid-height lateral deflection curves and failure modes with buckling in both the ‘C’ and ‘reverse C’ orientations, were fully reported. Following the testing programme, a numerical modelling programme was carried out and included a validation study, where FE models were developed and validated against the test results, and a parametric study, where the validated FE models were used to generate further numerical data over a wide range of cross-section dimensions, member effective lengths and loading combinations. The obtained test and numerical data were adopted to assess the accuracy and consistency of the relevant design interaction formulae, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001). The assessment results generally indicated that EN 1993-1-4 (CEN, 2015) leads to rather scattered resistance predictions, with many unsafe resistance predictions for Class 1 and 2 press-braked stainless steel channel section beam-columns but overly conservative resistance predictions for Class 3 press-braked stainless steel channel section beam-columns, although the resulting overall design accuracy is good. In comparison with EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) lead to much more consistent resistance predictions for each buckling orientation (regardless of cross-section sizes); in terms of the design accuracy, the ASCE (or AS/NZS) resistance predictions are slightly more accurate for press-braked stainless steel channel section beam-columns with failure in the ‘C’ orientation but much more conservative for those ‘reverse C’-orientation buckling cases.

CHAPTER 9

CONCLUSIONS AND SUGGESTIONS FOR FUTURE WORK

9.1 Conclusions

In this chapter, the key research findings and conclusions of this thesis are summarised, while more detailed concluding remarks are provided at the end of each chapter. Suggestions for future research are described thereafter.

The local buckling behaviour and cross-section resistances of press-braked stainless steel angle and channel section stub columns and beams have been experimentally and numerically studied in Chapter 3. An experimental programme, including 8 stub column tests and 10 in-plane bending tests (about the geometric axes for angle sections and minor principal axes for channel sections), has been firstly conducted, followed by a numerical modelling programme. It is worth highlighting that these stub column and beam tests on press-braked stainless steel angle and channel sections were the first to be reported (Zhang et al., 2020a). On the basis of the test and numerical results, the accuracy of the EC3 slenderness limits for classification of

press-braked stainless steel outstand and internal plate elements and the codified local buckling design provisions in predicting cross-section resistances have been assessed. The EC3 slenderness limits are safe when used for the classification of press-braked stainless steel equal-leg angle sections and plain channel sections, while the codified local buckling design provisions were shown to result in overly conservative and scattered design cross-section resistances, mainly due to the neglect of the favourable material strain hardening of stainless steel in the design. The accuracy of the advanced continuous strength method (CSM) (Afshan and Gardner, 2013; Zhao et al., 2017; Zhao and Gardner, 2018) has also been evaluated. The CSM allows proper exploitation of the material strain hardening and results in substantially more accurate and consistent cross-section resistance predictions over the existing design codes (CEN, 2015; ASCE, 2002; AS/NZS, 2001).

Experimental and numerical studies of press-braked stainless steel channel sections under combined loading have been carried out, as reported in Chapters 4 and 5. The testing programme includes two parts: (i) ten eccentric compression tests on press-braked stainless steel channel sections about the major principal axes and (ii) ten eccentric compression tests on two press-braked stainless steel channel sections about the minor principal axes. It is worth highlighting that these combined loading tests on press-braked stainless steel channel sections were the first around the world to be reported (Zhang et al., 2021c, 2021d). On the basis of the obtained test and numerical results, the accuracy and consistency of the codified design cross-section interaction curves, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001), have been evaluated for the first time. The evaluation results revealed that all the three examined design standards lead to unduly conservative and scattered failure load predictions for press-braked stainless steel channel stub columns under combined loading, principally owing to the adoption of linear interaction design curves anchored to conservative end points calculated without considering material strain hardening. New, more efficient, nonlinear design interaction curves anchored to more accurate CSM end points have been developed and shown to offer a much higher level of design accuracy and consistency over their codified counterparts. These research findings and new design approaches will serve as a basis for future revisions of EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001).

In Chapter 6, the flexural-torsional buckling behaviour and resistances of fixed-ended press-braked stainless steel equal-leg angle section columns have been experimentally and numerically investigated for the first time around the world. The testing programme includes initial global and torsional geometric imperfection measurements and fixed-ended compression tests on 16 press-braked stainless steel equal-leg angle section column specimens, which are the first such tests around the world (Zhang et al., 2019a). The experimental observations have revealed that flexural-torsional buckling of fixed-ended press-braked stainless steel angle section columns is length-dependent and always accompanied by minor-axis flexural buckling. All the existing design codes (CEN, 2015; ASCE, 2002; AS/NZS, 2001) do not consider the length-dependent characteristic of flexural-torsional buckling and the interaction of flexural-torsional buckling with minor-axis flexural buckling, and lead to conservative and scattered flexural-torsional buckling resistance predictions. A new design method has been proposed. Specifically, a series of flexural-torsional strength curves are firstly derived for stainless steel angle section columns to capture the length-dependence of flexural-torsional buckling and the interaction from minor-axis flexural buckling are then considered through combining the derived flexural-torsional strength curves with a new global strength curve. The new proposed design method is shown to yield accurate, consistent and safe flexural-torsional buckling resistance predictions for fixed-ended press-braked stainless steel angle section columns. These research findings and new design method will serve as a basis for future revisions of EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001).

Chapter 7 presents an in-depth experimental and numerical investigation into the minor-axis flexural buckling behaviour and resistances of press-braked stainless steel channel section columns. A total of 12 pin-ended column tests on press-braked stainless steel channel section columns have been conducted and reported for the first time (Zhang et al., 2020e). The obtained test and numerical data have been adopted to assess the accuracy of the relevant design column buckling curves, as set out in EN 1993-1-4 (CEN, 2015), SEI/ASCE 8-02 (ASCE, 2002) and AS/NZS 4673 (AS/NZS, 2001) and SCI P413 (SCI, 2017). Overall, the assessment results revealed that (i) the EC3 design buckling curve leads to many unsafe

flexural buckling resistance predictions for press-braked stainless steel channel section columns with member non-dimensional slendernesses less than around 1.0, (ii) the ASCE design buckling curve overpredicts the flexural buckling resistance for press-braked stainless steel channel section columns over the whole considered range of member non-dimensional slendernesses up to 3.0 and (iii) the design buckling curves set out in AS/NZS 4673 (AS/NZS, 2001) and SCI P413 (SCI, 2017) yield more conservative flexural buckling resistance predictions than those specified in EN 1993-1-4 (CEN, 2015) and SEI/ASCE 8-02 (ASCE, 2002), but with significantly reduced number of unsafe resistance predictions.

The world's first experimental and numerical investigations into the global instability and resistances of press-braked stainless steel channel section beam-columns under combined compression and bending moment have been reported in Chapter 8 (Zhang et al., 2021a). The testing programme includes ten eccentric compression tests and two types of failure mode, namely 'C'-orientation buckling and 'reverse C'-orientation buckling have been observed upon testing for press-braked stainless steel channel section beam-columns. On the basis of the obtained experimental and numerical data, the accuracy of the codified design interaction curves, as given in the European code EN 1993-1-4 (CEN, 2015), American specification SEI/ASCE 8-02 (ASCE, 2002) and Australian/New Zealand standard AS/NZS 4673 (AS/NZS, 2001), have been evaluated. The evaluation results revealed that the European code yields many unsafe resistance predictions for Class 1 and 2 press-braked stainless steel channel section beam-columns but overly conservative resistance predictions for Class 3 press-braked stainless steel channel section beam-columns, although the resulting overall design accuracy is good, but with a high level of scatter of the resistance predictions. Compared with the European code, the American specification and Australian/New Zealand standard result in slightly more accurate resistance predictions for press-braked stainless steel channel section beam-columns with buckling in the 'C' orientation, but lead to significantly more conservative predicted resistances for press-braked stainless steel channel section beam-columns with buckling in the 'reverse C' orientation. In terms of the design consistency, the American specification and Australian/New Zealand standard are shown to result in more consistent resistance predictions (regardless of cross-section dimensions and buckling orientations) than their Eurocode counterpart.

In summary, the cross-section and member behaviour and resistances of press-braked stainless steel angle and channel section structural components under isolated and combined loadings have been studied and reported in this thesis. The design rules specified in the existing international design codes and other relevant design approaches have been discussed and assessed, with their shortcomings highlighted. Moreover, new design methods for fixed-ended press-braked stainless steel angle section columns prone to flexural-torsional buckling and press-braked stainless steel channel sections under combined loading have been proposed, and shown to provide significantly improved levels of design accuracy and consistency. The reliability of the new proposed design methods has also been demonstrated through statistical analyses according to EN 1990 (CEN, 2002). These research findings and new design proposals presented in this thesis will serve as a basis for future revisions of international design codes for stainless steel structures.

9.2 Suggestions for future work

On the basis of the research work presented in this thesis, suggestions for future investigations into stainless steel angle and channel section structural members are given herein. While the focus of the present research is on the ambient temperature behaviour of stainless steel angle and channel section structural members, future research work are required to study their fire and post-fire behaviour.

The flexural-torsional buckling behaviour of angle section columns is influenced by the boundary conditions. For example, the interaction between flexural-torsional buckling and minor-axis flexural buckling is stronger for pin-ended boundary conditions in comparison with that for fixed-ended boundary conditions (as adopted in this thesis). Future research could be conducted to investigate the flexural-torsional buckling behaviour of pin-ended press-braked stainless steel angle section columns.

The in-plane bending behaviour of press-braked stainless steel channel section beams and minor-axis flexural buckling behaviour of press-braked stainless steel channel section columns have been studied in this thesis. Future experiments are required to study the lateral-torsional buckling behaviour of channel section beams and flexural-torsional buckling behaviour of channel section columns.

Finally, in view of the fact that stainless steel has excellent ductility, research into stainless steel angle and channel section connections (e.g., angle/channel-to-gusset plate connections) and frames requires more attention. Laboratory experiments and numerical simulations could be conducted to study their behaviour under static and cyclic loadings.

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