Behaviour and design of high-performance steel welded I-section structural components

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**NANYANG
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SINGAPORE

**BEHAVIOUR AND DESIGN OF HIGH-
PERFORMANCE STEEL WELDED I-SECTION
STRUCTURAL COMPONENTS**

SUN YAO

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

2021

Behaviour and Design of High-Performance Steel Welded I-Section Structural Components

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School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University
in partial fulfilment of the requirements 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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Supervisor Declaration Statement

I have reviewed the content and presentation style of this thesis and declare it is free of plagiarism and of sufficient grammatical clarity to be examined. To the best of my knowledge, the research and writing are those of the candidate except as acknowledged in the Author Attribution Statement. I confirm that the investigations were conducted in accord with the ethics policies and integrity standards of Nanyang Technological University and that the research data are presented honestly and without prejudice.

03-05-2021

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03-05-2021

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Zhao Ou

Authorship Attribution Statement

This thesis contains material from 8 papers published in the following peer-reviewed journals in which I am listed as an author.

Chapter 3 is published as Y. Sun, and O. Zhao. Material response and local stability of high-chromium stainless steel welded I-sections. *Engineering Structures* **178**, 212-226 (2019). DOI: 10.1016/j.engstruct.2018.10.024.

The contributions of the co-authors are as follows:

- Dr Zhao provided the initial project direction.
- I prepared the manuscript draft. The manuscript was revised by Dr Zhao.
- I co-designed the study with Dr Zhao and performed all the laboratory work at the School of Civil and Environmental Engineering. I also analysed the data.

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(i) Y. Sun, Y. Liang, and O. Zhao. Testing, numerical modelling and design of S690 high strength steel welded I-section stub columns. *Journal of Constructional Steel Research* **159** 521-533 (2019). DOI: 10.1016/j.jcsr.2019.05.014.

(ii) Y. Sun, A. He, Y. Liang, and O. Zhao. In-plane bending behaviour and capacities of S690 high strength steel welded I-section beams. *Journal of Constructional Steel Research* **162** 105741 (2019). DOI: 10.1016/j.jcsr.2019.105741.

The contributions of the co-authors are as follows:

- Dr Zhao provided the initial project direction.
- I prepared the manuscript draft, which was revised by Dr Liang and Dr Zhao.
- I co-designed the study with Dr Zhao and performed all the laboratory work at the School of Civil and Environmental Engineering. I also analysed the data.
- Dr Liang provided guidance in the interpretation of the experimentally measured data.

- Dr He assisted in preparing the beam test setup and helped me to analyse the test results.

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(i) Y. Sun, A. He, Y. Liang, and O. Zhao. Flexural buckling behaviour of high-chromium stainless steel welded I-section columns. *Thin-Walled Structures* **154** 106812 (2020). DOI: 10.1016/j.tws.2020.106812.

(ii) Y. Sun, Y. Liang, and O. Zhao. Local–flexural interactive buckling behaviour and resistances of high-chromium stainless steel slender welded I-section columns. *Engineering Structures* **220** 111022 (2020). DOI: 10.1016/j.engstruct.2020.111022.

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- I prepared the manuscript draft. It was then revised by Dr Liang and Dr Zhao.
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- Dr He helped to perform the parametric studies.

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- Ms Jiang assisted in preparing the beam-column test setup and helped me to analyse the test results.
- Dr Liang assisted in the numerical modelling programme.

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ABSTRACT

High-performance steels, including stainless steels and high strength steels, have been gaining increasing attention in civil engineering, owing to their appealing sustainable and mechanical advantages. The present thesis is focusing on the behaviour and design of high-performance steel welded I-section structural components. A comprehensive testing programme was firstly conducted, which adopted twelve welded I-sections made of two types of high-performance steel (grade EN 1.4420 high-chromium stainless steel and S690 high strength steel). Material testing was carried out to derive the material stress–strain responses of the two adopted high-performance steel grades, and membrane residual stress measurements were performed to determine the membrane residual stress distributions and amplitudes in the studied high-performance steel welded I-sections. At the cross-sectional level, a total of 24 axially loaded stub column tests, 20 in-pane four-point bending tests, and 10 eccentrically loaded stub column tests were conducted to investigate the cross-section resistances and local stability of high-performance steel welded I-sections. At the member level, 40 column tests and 10 beam-column tests were performed to study the load-carrying capacities and member instability of high-performance steel welded I-sections. The testing programme was supplemented by a numerical modelling programme, where finite element models were firstly developed and validated against the test results and then employed to perform a series of numerical parametric studies to generate additional numerical data over a wide range of cross-section dimensions, member lengths and loading combinations. The obtained test and numerical results were used to evaluate (i) the applicability of the current design codes for normal stainless steel members to the new high-chromium stainless steel welded I-section members and (ii) the accuracy of the existing relevant design codes for S690 high strength steel welded I-section members. The evaluation results generally indicated that the current design codes result in inaccurate resistance

predictions, mainly owing to the lack of due consideration of the specific material characteristics of the two types of high-performance steel. New design methods have also been proposed, which were shown to provide substantially improved design accuracy and consistency over the existing design standards. The reliability of the proposed design methods has been confirmed by means of statistical analyses, demonstrating their suitability for incorporation into future revisions of international design codes for stainless steel and high strength steel structures.

LIST OF PUBLICATIONS

1. Sun, Y. and Zhao, O. (2019) Material response and local stability of high-chromium stainless steel welded I-sections. *Engineering Structures*, 178, 212–226.
2. Sun, Y., Liang, Y. and Zhao, O. (2019) Testing, numerical modelling and design of S690 high strength steel welded I-sections stub columns. *Journal of Constructional Steel Research*, 159, 521–533.
3. Sun, Y., Liu, Z., Liang, Y. and Zhao, O. (2019) Experimental and numerical investigation of hot-rolled austenitic stainless steel equal-leg angle sections. *Thin-Walled Structures*, 144, 106225.
4. Sun, Y., He, A., Liang, Y. and Zhao, O. (2019) In-plane bending behaviour and capacities of S690 high strength steel welded I-sections beams. *Journal of Constructional Steel Research*, 162, 105741.
5. Su, A., Sun, Y., Liang, Y. and Zhao, O. (2020) Material properties and membrane residual stresses of S690 high strength steel welded I-sections after exposure to elevated temperatures. *Thin-Walled Structures*, 152, 106723.
6. Sun, Y., He, A., Liang, Y. and Zhao, O. (2020) Flexural buckling behaviour of high-chromium stainless steel welded I-section columns. *Thin-Walled Structures*, 154, 106812.
7. Sun, Y., Liang, Y. and Zhao, O. (2020) Local–flexural interactive buckling behaviour and resistances of high-chromium stainless steel slender welded I-section columns. *Engineering Structures*, 220, 111022.
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9. Sun, Y., Liang, Y., Zhao, O. and Young, B. (2021) Experimental and numerical investigations of S690 high strength steel welded I-sections under combined compression and bending. *Journal of Structural Engineering (ASCE)*, 147(5), 04021054.
10. Su, A., Sun, Y., Liang, Y. and Zhao, O. (2021) Membrane residual stresses and local buckling of S960 ultra-high strength steel welded I-section stub columns. *Thin-Walled Structures*, 161, 107497.
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12. Zhong, Y., Sun, Y., Tan, K.H. and Zhao, O. (2021) Testing, modelling and design of high strength concrete-filled high strength steel tube (HCFHST) stub columns under combined compression and bending. *Engineering Structures*, 241, 112334.
13. He, A., Sun, Y., Wu, N. and Zhao, O. (2021) Testing, modelling and design of eccentrically loaded austenitic stainless steel CHS stub columns after exposure to elevated temperatures. *Thin-Walled Structures*, 107885. DOI: 10.1016/j.tws.2021.107885.

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LIST OF SYMBOLS

A	Gross cross-section area
A_e	AISC 27 effective cross-section area
A_{eff}	Effective cross-section area
$A_{eff,AISC}$	AISC effective cross-section area
$A_{eff,AS}$	AS 4100 effective cross-section area
$A_{eff,EC3}$	Eurocode 3 effective cross-section area
a, b, c, d	Distribution parameters of residual stress predictive model
a_w	web-to-gross area ratio
b	Mean ratio of test and numerical to design model resistances
b_f	Flange width of I-section
b_p	AISC 27 flat width of outstand flange
b_s	Distance between strain gauges attached to welded I-section
C_1, C_2, C_3, C_4	CSM material parameter
c	Flat width of plate element
c_{eff}	Effective width of plate element
c_f	Flat width of flange
c_w	Flat width of web
D_L	Vertical deflection at loading point in four-point bending tests
D_M	Vertical deflection at mid-span in four-point bending tests
d_s	Distance between strain gauge and adjacent flange edge
E	Young's modulus
E_{sh}	Strain hardening slope
E_t	Tangent modulus at yield stress
$E_{0.2}$	Tangent modulus at 0.2% proof stress
e_0	Initial loading eccentricity
f or σ	Stress

f_{fc}	Peak compressive residual stress in flange of high strength steel welded I-section
f_{ft}	Peak tensile residual stress in flange of high strength steel welded I-section
f_{wc}	Peak compressive residual stress in web of high strength steel welded I-section
f_{wt}	Peak tensile residual stress in web of high strength steel welded I-section
f_u	Ultimate stress of high strength steel
f_y	Yield stress of high strength steel
f_u/f_y	Ultimate-to-yield tensile strength ratio of high strength steel
h	Outer height of I-section
h_w	Clear distance between flanges
I	Second moment of area
k_c	ANSI/AISC 360-16 geometric factor
$k_{yy}, k_{yz}, k_{zy}, k_{zz}$	Eurocode interaction factor
k_z	EN 1993-1-4 interaction factor
$k_{d,n}$	Design fractile factor
k_σ	EN1993-1-5 buckling factor
L	Member length
L_e	Member effective length
L_m	Distance between the loading points in four-point bending tests
L_0	Nominal length between gauge holes
$M_{N,y}$	Reduced cross-section major-axis plastic moment resistance
$M_{N,z}$	Reduced cross-section minor-axis plastic moment resistance
M_R	Cross-section bending moment resistance
M_{cy}	AISC cross-section bending moment resistance about major axis
M_{cz}	AISC cross-section bending moment resistance about minor axis

M_{eff}	Effective cross-section bending moment capacity
$M_{eff,EC3}$	EN 1993-1-12 effective cross-section bending moment capacity
M_{el}	Cross-section elastic moment capacity
$M_{el,y}$	Cross-section elastic moment capacity about major axis
$M_{el,z}$	Cross-section elastic moment capacity about minor axis
M_{pl}	Cross-section plastic moment capacity
$M_{pl,y}$	Cross-section plastic moment capacity about major axis
$M_{pl,z}$	Cross-section plastic moment capacity about minor axis
$M_{s,y}$	AS cross-section bending moment resistance about major axis
$M_{s,z}$	AS cross-section bending moment resistance about minor axis
M_u	Failure bending moment
$M_{u,AISC}$	AISC design cross-section bending moment resistance
$M_{u,EC3}$	EC3 design cross-section bending moment resistance
$M_{u,CSM}$	CSM design cross-section bending moment resistance
$M_{u,pred}$	Design ultimate cross-section bending moment resistance
$M_{u,y}$	Design cross-section major-axis failure moment
$M_{u,z}$	Design cross-section minor-axis failure moment
$M_{u,1st,el}$	First-order elastic failure bending moment at mid-height
$M_{u,2nd,el}$	Second-order elastic failure bending moment at mid-height
$M_{u,2nd,incl}$	Second-order inelastic failure bending moment at mid-height
$M_{z,Rd}$	Cross-section minor-axis bending moment resistance
m	Ramberg–Osgood exponent
$m_{1.0}$	Compound Ramberg–Osgood exponent based on $\sigma_{0.2}$ and $\sigma_{1.0}$
N_R	Column flexural buckling resistance
$N_{b,z,Rd}$	EN 1993-1-4 column minor-axis flexural buckling resistance
N_{cr}	Euler buckling load
N_u	Failure load
$N_{u,AISC}$	AISC design compression resistance

$N_{u,EC3}$	EC3 design compression resistance
$N_{u,csm}$	CSM design compression resistance
$N_{u,pred}, N_{u,Ed}$	Design ultimate compression resistance
N_y	Cross-section yield load
n	Ramberg–Osgood exponent or ratio of design axial compression load to cross-section yield load
P_m	Professional factor
Q_a	AISC 27 net section reduction factor for slender stiffened plate elements
Q_s	AISC 27 net section reduction factor for slender unstiffened plate elements
R	Cross-section rotation capacity
r_1	Reading of Demec gauge taken before sectioning
r_2	Reading of Demec gauge taken after sectioning
t	Plate thickness of I-section
t_f	Flange thickness of I-section
t_w	Web thickness of I-section
t_1	Length of temperature reference bar before sectioning
t_2	Length of temperature reference bar after sectioning
V_Q	COV of load effects
V_R	COV of resistances
V_r	Combined COV incorporating both model and basic variable uncertainties
V_δ	COV of test and numerical resistances relative to resistance model
W_{eff}	Effective section modulus
$W_{eff,EC3}$	EN 1993-1-12 effective section modulus
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
α	CSM bending parameter, Eurocode 3 imperfection parameter or AISC amplification factor
α_a	AS 4100 compression member factor
α_b	AS 4100 member section constant
α_c	AS 4100 flexural buckling reduction factor
β	Reliability index
χ	Eurocode 3 reduction factor for global buckling
Δ	Mid-height lateral deflection of member
Δ_u	Mid-height lateral deflection of member at failure load
δ	Maximum deviation of curved strip
δ_u	End shortening at failure
ε	Strain or Eurocode 3 material parameter
ε_{AISC}	ANSI/AISC 360-16 material parameter
ε_{AS}	AS 4100 material parameter
ε_c	Corrected relieved strain
ε_{csm}	CSM predicted failure strain of cross-section
ε_{EC3}	EN1993-1-12 material parameter
ε_f	Fracture strain measured over standard gauge length of 50 mm
ε_{ln}^{pl}	Logarithmic plastic strain
ε_{nom}	Engineering strain
ε_u	Strain at ultimate stress
ε_y	Material yield strain
ε_0	Relieved strain

γ_{M0}	Partial safety factor for cross-section resistance
γ_{M1}	Partial safety factor for member resistance
η	AS 4100 compression member imperfection factor
κ	Curvature
κ_{pl}	Elastic curvature corresponding to M_{pl}
κ_u	Curvature when falling branch of moment–curvature curve reaches M_{pl}
$\bar{\lambda}_{AISC}$	AISC member non-dimensional slenderness
$\bar{\lambda}_{AS}$	AS 4100 member non-dimensional slenderness
$\bar{\lambda}_{EC3}$	Eurocode 3 member non-dimensional slenderness
$\bar{\lambda}_l$	Plate element slenderness
λ_n	AS 4100 modified member slenderness
$\bar{\lambda}_p$	Cross-section slenderness
λ_{pf}	AISC compact limiting width-to-thickness ratio for flange
λ_{rf}	AISC non-compact limiting width-to-thickness ratio for flange
$\bar{\lambda}_{r,int}$	AISC limiting width-to-thickness ratio for internal plate elements
$\bar{\lambda}_{r,out}$	AISC limiting width-to-thickness ratio for outstand plate elements
λ_{sp}	AS plasticity limits for most slender constituent plate element
λ_{sy}	AS yield limits for most slender constituent plate element
$\bar{\lambda}_0$	Eurocode 3 limiting slenderness for global instability
$\omega_{D\&W}$	Initial local imperfection predicted by Dawson and Walker model
ω_{f1}, ω_{f2}	Maximum initial local imperfection magnitudes of two flanges
ω_g	Initial global geometric imperfection magnitude of specimen
ω_n	Maximum local imperfection magnitude from measured line n
ω_w	Maximum initial local imperfection magnitude of web

ω_0	Initial local geometric imperfection magnitude of specimen
ϕ	Eurocode 3 buckling parameter
ϕ_{AISC}	AISC resistance factor
ρ	Eurocode 3 reduction factor for plate width
ρ_{AS}	AS 4100 reduction factor
σ_{cr}	Elastic critical local buckling stress of cross-section
σ_{fc}	Peak compressive residual stress in flange of stainless steel welded I-section
σ_{ft}	Peak tensile residual stress in flange of stainless steel welded I-section
σ_{nom}	Engineering stress
σ_{true}	True stress
σ_u	Ultimate stress of stainless steel
σ_{wc}	Peak compressive residual stress in web of stainless steel welded I-section
σ_{wt}	Peak tensile residual stress in web of stainless steel welded I-section
$\sigma_{0.2}$	0.2% proof stress
$\sigma_{1.0}$	1.0% proof stress
θ	Angle parameter
ζ	AS 4100 buckling parameter

CHAPTER 1

INTRODUCTION

1.1 Background

High-performance steel is the designation given to steels that offer higher performances in areas such as mechanical strength, corrosion and fire resistance and ductility, in comparison with conventional carbon steels. Stainless steel and high strength steel are the two most commonly used high-performance steels in civil and offshore engineering in the past two decades (Gardner, 2005, 2019; IABSE, 2005; Pocock, 2006; Baddoo, 2008; Gedge, 2008).

Stainless steel is a corrosion-resistant alloy with a minimum of 10.5% chromium content by mass, which is the key to its crucial corrosion resistance property. Together with chromium, nickel, molybdenum, manganese, titanium and copper are the main alloying elements in stainless steels, which stabilise the crystal microstructure and contribute to a number of positive attributes of stainless steels, such as ductility, machinability and high temperature performance. Stainless steels are grouped into five main families according to their metallurgical structure, namely austenitic, ferritic, duplex, martensitic and precipitation hardening, of which the first three are used for structural applications (Gardner 2005). In comparison with carbon steels, stainless steels possess substantially better resistance against corrosion and are thus more durable, resulting in negligible maintenance costs during the service life of structures, and are 100% recyclable after use, with no degradation of mechanical and physical properties. High strength steel (HSS), with the nominal yield stress equal to or greater than 460 MPa, has superior mechanical strength, providing the possibility

of designing structural members with small cross-section dimensions and light self-weights; therefore, the use of high strength steel can lead to great savings in foundation costs and also facilitate fabrication, handling, transportation and assembly of the corresponding structural members.

Although the favourable mechanical properties make stainless steel and high strength steel promising construction materials, their actual use in engineering applications is limited by (i) the high initial material cost and (ii) the lack of efficient design rules. New high-performance steels with low material price but still favourable mechanical properties are continuously being developed; a recent example is grade EN 1.4420 high-chromium austenitic stainless steel, which has higher material strengths and corrosion resistance but with a lower material price. Moreover, most of the design rules in the existing design standards for high-performance steel structures, including European codes (CEN, 2007, 2015), American specifications (AISC, 2013, 2016) and Australian standard (AS, 2016), were developed by directly mirroring those for normal strength carbon steel structures (CEN, 2014; AISC, 2016; AS, 2016), with no rational consideration of the distinctive material characteristics (e.g., strain hardening) of high-performance steels, which have been found to be unduly inaccurate; new design methods for high-performance steel structures with improved design accuracy and efficiency were thus required. Experimental studies on different types of high-performance steel members subjected to various types of loading have been prompted, in order to verify their structural behaviour and resistances, evaluate the accuracy of the codified design rules and develop new accurate and efficient design methods. The current study focuses on the structural behaviour and design of welded I-section members made of grade EN 1.4420 high-chromium stainless steel and grade S690 high strength steel.

1.2 Structural applications

In recent years, stainless steel and high strength steel have been gaining increasing use in civil and offshore engineering. Early structural applications of stainless steel include the supporting structures of the St Paul's Cathedral in London in 1925, the cladding of the Chrysler Building in New York in 1929 and the exterior surface of the Gateway Arch in St Louis, Missouri in 1963. More recently, stainless steel has been used to construct primary load-bearing components in numerous projects. Figures 1.1 and 1.2 show two examples of stainless steel structures, including the Helix pedestrian bridge in Singapore and Apollo road bridge in Slovakia, respectively, both of which indicate the increasing use of stainless steel as a construction material rather than simply a decorative material. Other stainless steel structural applications have been summarised in Gardner (2008a), Gedge (2008) and Baddoo (2013).

Figure 1.1: Helix pedestrian bridge, Singapore

Figure 1.2: Apollo road bridge, Slovakia

The structural use of high strength steel began in the 1960s and has been gradually adopted throughout the world, particularly in Japan, Germany, China and the United States. In the Millau Viaduct (Figure 1.3), S460ML high strength steel was adopted for the deck and pylons. The Landmark Tower (Figure 1.4) in Japan was constructed using S690 high strength steel welded I-sections as columns to reduce the self-weights and thus the seismic reaction forces of the structure under earthquake loading. In these long-span structures and high-rise buildings, high strength steel has been effectively exploited, bringing important structural and economic benefits.

Figure 1.3: Millau Viaduct, France

(<https://www.lafargeholcim.com/millau-viaduct-lafargeholcim-cement-and-concrete-worlds-highest-bridge>)

Figure 1.4: Landmark Tower, Japan
(https://en.wikipedia.org/wiki/Yokohama_Landmark_Tower)

1.3 Research objectives

Given the substantially higher material price of high-performance steel (stainless steel and high strength steel) in comparison with that of conventional carbon (normal strength) steel, structural design efficiency is of primary concern. However, most of the provisions and formulae set out in the existing design standards for stainless steel and high strength steel welded I-sections were developed by directly mirroring those for their conventional carbon steel counterparts, with an unduly low level of design efficiency. Therefore, investigations of high-performance steel welded I-sections and development of efficient design rules are the objectives of the present thesis.

1.4 Thesis outline

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

This chapter presents a brief introduction to the background of high-performance steel (stainless steel and high strength steel) and its actual applications in civil engineering. The research objectives of this thesis are also reported.

Chapter 2 provides an extensive review of previous studies that are pertinent to this thesis, with the aim of offering an overview of important topics, while specific details from the literature will be further explained in the following chapters.

Chapter 3 and Chapter 4 describe thorough experimental and numerical investigations into the material responses and local stability of welded I-sections made of new grade EN 1.4420 high-chromium stainless steel and grade S690 high strength steel subjected to isolated loadings. In each chapter, the experimental and numerical programme on stub columns (in pure compression) and beams (in pure bending) is fully reported. The experimentally and numerically obtained results are employed to evaluate the applicability of the current design methods to high-chromium stainless steel welded I-sections (Chapter 3) and to assess the accuracy of the design standards for S690 high strength steel welded I-sections (Chapter 4).

Chapter 5 reports a comprehensive testing and numerical modelling programme on the cross-section behaviour and resistances of S690 high strength steel welded I-sections under combined compression and bending moment. Based on the obtained experimental and numerical results, the accuracy of the design interaction formulae, as set out in the European, American and Australian standards, for S690 high strength steel welded I-sections under compression is evaluated.

Chapter 6 and Chapter 7 experimentally and numerically investigate the buckling behaviour (minor-axis flexural buckling and local–flexural interactive buckling) of grade EN 1.4420 high-chromium stainless steel and grade S690 high strength steel welded I-section columns, respectively. The experimental and numerical results are fully analysed and then adopted to evaluate the applicability of the codified design

rules to high-chromium stainless steel welded I-section columns (Chapter 6) and to assess the accuracy of the codified design rules for S690 high strength steel welded I-section columns (Chapter 7). New improved design approaches are also proposed.

Chapter 8 presents experimental and numerical studies of the global instability of high-chromium stainless steel welded I-section beam-columns under combined compression and minor-axis bending moment. A comprehensive testing programme, in conjunction with a systematic numerical modelling programme, is reported. The codified beam-column interaction curves are described and then evaluated through comparisons against the experimental and numerical results. An improved design interaction curve is then proposed.

Chapter 9 concludes the important findings of this research and provides suggestions for future research work.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter presents a broad review of previous research work that is relevant to this thesis. The development of international design standards for high-performance steel structures, including the European codes, American specifications and Australian standards, is firstly reviewed. Previous relevant studies on stainless steel and high strength steel welded I-section structural components have been summarised and presented. This is followed by key research findings on finite element simulations of high-performance steel welded I-section structural components. Further relevant studies on specific topics are reviewed in the following chapters.

2.2 Design standards for stainless steel structures

The earliest design standard for structural stainless steel, referred to as “Specification for the Design of Light Gauge Cold-Formed Stainless Steel Structure Members” (AISI, 1962), was issued by the American Iron and Steel Institute (AISI). The current American specifications for structural stainless steel are “Specification for the Design of Cold-formed Stainless Steel Structural Members” (ASCE, 2002) released by the American Society of Civil Engineers (ASCE) in 2002 and “Design Guide 27: Structural Stainless Steel” (AISC, 2013) issued by the American Institute of Steel Constructors (AISC) in 2013.

In 1994, Euro Inox (1994) published the first European stainless steel design guidance, which is known as “Design Manual for Structural Stainless Steel”. Based on this guidance, the European pre-standard for structural stainless steel, referred to as “ENV 1993-1-4: Design of steel structures, Supplementary rules for stainless steels” was issued in 1996 by the European Standards Committee (CEN), and then developed to a formal European code – EN 1993-1-4 in 2006, which was revised to the current version (CEN, 2015) in 2015.

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, which was established primarily based on the American specification, despite some different design rules. The detailed description of the development of the Australian/New Zealand standard (AS/NZS, 2001) was reported in Rasmussen and Gough (2002).

Although several design standards have been developed for stainless steel structures, only the European code 1993-1-4 (CEN, 2015) and American design guide AISC 27 (AISC, 2013) provide design rules for stainless steel welded I-section structural components, which are evaluated in this thesis.

2.3 Design standards for high strength steel structures

The design guidelines for high strength steel structures are limited. The current European code EN 1993-1-12 (CEN, 2007) was published in 2007 specifically for the design of high strength steels with grades greater than S460 up to S700, with most of the design rules directly mirroring those for normal strength steels. The existing American specification, referred to as “Specification for Structural Steel Buildings” (AISC, 2016), was released by the American Institute of Steel Construction (AISC) in 2013 and revised in 2016 and specific for high strength steel structures with

material grades up to grade S690. In Australia, the design standard AS 4100 (AS, 1998) was developed in 1998 and originally covered steels with the nominal yield stresses up to 450 MPa, which was then revised and extended the scope of application to grade S690 in 2016.

2.4 Previous research on high-performance steel welded I-sections

2.4.1 Material response

2.4.1.1 Material response of stainless steel

Conventional 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. The nonlinear stress–strain response of stainless steel can be represented by the two-stage Ramberg–Osgood (R–O) material model, which is an extension of the basic R–O material model (Ramberg and Osgood, 1943; Hill, 1944), developed by Mirambell and Real (2000), Rasmussen (2003), Gardner and Ashraf (2006) and Arrayago et al. (2015). The first and second stages of the two-stage R–O material model, respectively adopted below and beyond the 0.2% proof stress $\sigma_{0.2}$, are defined by Equations (2.1) and (2.2), where ε and σ are the strain and stress, respectively, $\sigma_{1.0}$ is the 1.0% proof stress, σ_u is the ultimate stress, E is the initial Young's modulus, $E_{0.2}$ is the tangent modulus at the 0.2% proof stress $\sigma_{0.2}$, and n and $m_{1.0}$ are the strain hardening exponents. The two-stage R–O material model provides excellent agreement with experimentally obtained stress–strain curves up to strains of approximately 10% and is thus suitable for practical numerical

modelling and design analysis (Gardner, 2005). A three-stage material model for stainless steel has recently been proposed by Quach et al. (2008) and Hradil et al. (2013), for simulating the structural behaviour of stainless steel joints and tension members, which require the full range of stress–strain curve up to the ultimate strain to be accurately defined. For austenitic stainless steels, the ductility, measured in terms of the strain at fracture, is around 40–60%, nearly two times that of carbon steels, while the ductility of duplex and ferritic stainless steels is approximately 30–50% and 15–30%, respectively.

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

$$\varepsilon = \frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \left(\varepsilon_{t,1.0} - \varepsilon_{t,0.2} - \frac{\sigma_{1.0} - \sigma_{0.2}}{E_{0.2}} \right) \left(\frac{\sigma - \sigma_{0.2}}{\sigma_{1.0} - \sigma_{0.2}} \right)^{m_{1.0}} + \varepsilon_{t,0.2} \quad (2.2)$$

for $\sigma_{0.2} \leq \sigma \leq \sigma_u$

The stainless steel grade studied in this thesis is a newly developed high-chromium austenitic stainless steel grade EN 1.4420. In comparison with its commonly used austenitic stainless steel counterparts such as grade EN 1.4301 and grade EN 1.4404, the new grade EN 1.4420 possesses higher nitrogen and chromium contents, leading to higher material strengths and better resistance against corrosion, but lower molybdenum and nickel contents, resulting in lower material price (Baddoo, 2008; CEN, 2016a; Liang et al., 2018). Table 2.1 presents the chemical compositions of the two commonly used austenitic stainless steel grades EN 1.4301 and EN 1.4404 and the new grade EN 1.4420 (CEN, 2016a), while their nominal mechanical properties and corrosion resistances are given in Table 2.2 (CEN, 2016a), where PREN (pitting resistance equivalent number) is a measure of the resistance of stainless steel against corrosion.

Table 2.1: Chemical compositions of stainless steel grades EN 1.4301, EN 1.4404 and EN 1.4420

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

Table 2.2: Nominal mechanical properties and corrosion resistances of stainless steel grades EN 1.4301, EN 1.4404 and EN 1.4420

Grade	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	PREN
1.4301	≥ 210	≥ 250	520 – 720	18
1.4404	≥ 220	≥ 260	520 – 670	24
1.4420	≥ 320	≥ 350	630 – 830	26

2.4.1.2 Material response of high strength steel

For high strength steels with grades up to S690, their stress–strain responses are similar to those of normal strength steels, i.e. a linear elastic response up to a clearly defined yield point, followed by a yield plateau and then some strain hardening, though with higher yield and ultimate stresses but lower ultimate and fracture strains. For high strength steels with grades beyond S690, the material stress–strain responses become increasingly nonlinear with reduced yield plateaus. Grade S690 high strength steel is studied in this thesis, with its chemical compositions and nominal material properties reported in Table 2.3 (CEN, 2009). Previous studies (Li et al., 2015; Wang et al., 2017; Ban & Shi, 2018) on the production routes of high strength steels highlighted that the ultimate tensile stress f_u is not increased at the same rate as the yield stress f_y ; as a result, the degree of material strain hardening, which is normally reflected by the ultimate-to-yield tensile stress ratio f_u/f_y , decreases with high strength steel grade. Fukumoto (1998) and Langenberg (2008) proposed empirical formulae to define the relationship between f_u/f_y and f_y , as given by Equation (2.3) and Equation (2.4), respectively.

$$f_u/f_y = 0.83 + 203.8/f_y \quad (2.3)$$

$$f_u/f_y = \left[1 - 0.72e^{(-0.0027f_y)} \right]^{-1} \quad (2.4)$$

Table 2.3: Chemical compositions and nominal material properties of grade S690 high strength steel

(a) Chemical compositions						
Grade	C (%)	Mn (%)	Cr (%)	N (%)	Mo (%)	Ni (%)
S690	≤0.20	≤1.70	≤1.50	≤0.015	≤0.70	≤2.00
(b) Nominal material properties						
Grade	f_y (N/mm ²)	f_u (N/mm ²)	ε_u (%)			
S690	690	770 – 940	12			

Several material constitutive models have been developed for representing the material stress–strain response of high strength steel, including the revised Ramberg–Osgood material model (Ramberg & Osgood, 1943; Shi et al., 2016), as given by Equation (2.5), and the multi-linear material model (Shi et al., 2016), as given by Equation (2.6), where ε and f are the strain and stress, respectively, E is the initial Young’s modulus, E_t is the tangent modulus at the yield stress f_y , ε_y and ε_u are the strains corresponding to the yield and ultimate stresses, ε_h is the strain corresponding to $(f_y+f_u)/2$, and m and n are the strain hardening exponents.

$$\varepsilon = \begin{cases} \frac{f}{E} + 0.002 \left(\frac{\sigma}{f_y} \right)^n & \text{for } f \leq f_y \\ \frac{f - f_y}{E_t} + \left(\varepsilon_u - \frac{f_u - f_y}{E_t} - \varepsilon_y \right) \left(\frac{f - f_y}{f_u - f_y} \right)^m + \varepsilon_y & \text{for } f > f_y \end{cases} \quad (2.5)$$

$$\varepsilon = \begin{cases} \frac{f}{E} & \text{for } f \leq 0.85 f_y \\ \frac{0.85 f_y}{E} + \frac{f - 0.85 f_y}{0.15 f_y} \left(\varepsilon_y - \frac{0.85 f_y}{E} \right) & \text{for } 0.85 f_y < f \leq f_y \\ \varepsilon_y + \frac{f - f_y}{0.5(f_u - f_y)} (\varepsilon_h - \varepsilon_y) & \text{for } 0.5(f_y + f_u) < f \leq f_u \\ \varepsilon_h + \frac{f - 0.5(f_u + f_y)}{0.5(f_u - f_y)} (\varepsilon_u - \varepsilon_h) & \text{for } f_y < f \leq 0.5(f_y + f_u) \end{cases} \quad (2.6)$$

2.4.1.3 Membrane residual stresses

The welding process introduces membrane residual stresses into welded steel sections, which can result in premature failure and reduced load-carrying capacities of structural members. The Swedish regulations BSK 99 (2003) and European convention ECCS (1976) provide predictive models for predicting distributions and amplitudes of membrane residual stresses in normal strength carbon steel welded I-sections. Bredenkamp et al. (1992), Lagerqvist and Olsson (2001), Yuan et al. (2014a) and Gardner et al. (2016) conducted a series of membrane residual stress measurements on stainless steel I-sections fabricated by traditional metal arc welding and advanced laser welding through using the sectioning method and developed several predictive models, which follow the same general pattern as that employed in BSK 99 (2003) and ECCS (1976), but with different distribution parameters and peak membrane residual stress magnitudes. The membrane residual stresses in S460 high strength steel welded I-sections were measured and reported in Ban et al. (2012) and Wang et al. (2012a) by means of the sectioning method and the hole-drilling method, respectively; on the basis of the measured data, two predictive models were proposed, with the adopted pattern being similar to that of the BSK and ECCS predictive models but lower peak membrane residual stress magnitudes and different distribution parameters. To date, research on membrane residual stresses in S690 high strength steel welded I-sections remains scarce, with the only study reported by Liu and Chung (2018). Detailed discussions and analyses of the codified and proposed membrane residual stress models are provided in Chapters 3 and 4.

2.4.2 Cross-section behaviour

2.4.2.1 Previous experimental investigations

Previous experimental investigations into the cross-section behaviour and resistances of high-performance steel welded I-section components are reviewed in this section. A series of stub column tests were conducted on high-performance steel welded I-

sections to study their local buckling behaviour and cross-section compression resistances, while three-point and four-point bending tests were performed to study the in-plane flexural behaviour and resistances of high-performance steel welded I-section beams subjected to a moment gradient and constant moment, respectively. Tables 2.4 and 2.5 summarise the previous stub column and beam tests on high-performance steel welded I-sections, respectively. Although extensive experimental studies have been performed on high-performance steel welded I-sections subjected to isolated loadings (i.e. pure compression and pure bending), investigations into their cross-section behaviour and resistances under combined compression and bending moment remain scarce.

Table 2.4: Summary of previous stub column tests on high-performance steel welded I-sections

Steel	Type and grade	Reference	No. of tests
Stainless steel	Austenitic EN 1.4301	ECSC (2001)	3
		Kuwamura (2003)	8
		Yuan et al. (2014b)	10
	Austenitic EN 1.4307	Gardner et al. (2016)	4
	Austenitic EN 1.4318	Kuwamura (2003)	8
	Austenitic EN 1.4404	Gardner et al. (2016)	2
	Austenitic EN 1.4571	Gardner et al. (2016)	3
	Duplex EN 1.4162	Saliba and Gardner (2013)	4
	Duplex EN 1.4462	ECSC (2000)	1
		Yuan et al. (2014b)	5
	Ferritic EN 1.4003	Bredenkamp and van den Berg (1995)	2
High strength steel	S440	Im et al. (2001,2005)	19
		Lee et al. (2015)	10
	S460	Shi et al. (2014)	9
	S650	Kim et al. (2012)	10
		Yoo et al. (2013)	7
	S690	Rasmussen and Hancock (1992)	6
	S960	Shi et al. (2015)	4
		Li et al. (2019)	4
		Su et al. (2021a)	16

Table 2.5: Summary of previous in-plane bending tests on high-performance steel welded I-sections

Steel	Type and grade	Reference	No. of tests
Stainless steel	Austenitic EN 1.4301	Burgan et al. (2000)	9
		ECSC (2001)	3
	Austenitic EN 1.4306	Real and Mirambell (2005)	2
	Austenitic EN 1.4307	Bu and Gardner (2018)	10
	Austenitic EN 1.4404	Bu and Gardner (2018)	6
	Austenitic EN 1.4571	Bu and Gardner (2018)	4
	Duplex EN 1.4162	Saliba and Gardner (2013)	8
	Duplex EN 1.4462	Burgan et al. (2000)	3
		ECSC (2001)	1
High strength steel	S550	Green et al. (2002)	10
	S650	Lee et al. (2013)	11
	S690	McDermott (1969)	9
		Beg and Hladinik (1996)	10

2.4.2.2 Codified design rules for local buckling

The current international standards adopt the cross-section classification framework and an elastic, perfectly plastic material model without considering material strain hardening for the design of high-performance steel welded I-sections susceptible to local buckling. The class of a welded I-section is defined based on its most slender constituent plate element without considering the favourable element interaction effect. For high-performance steel welded I-sections in bending, the European codes EN 1993-1-4 (CEN, 2015) and EN 1993-1-12 (CEN, 2007) specify four cross-section classes: Class 1 and 2 (plastic) sections in bending can achieve the plastic moment capacities M_{pl} , Class 3 (elastic) sections in bending are capable of attaining the elastic moment capacities M_{el} and Class 4 (slender) sections fail before the material yield stresses are attained, with the design bending moment resistances limited to the effective moment capacities M_{eff} . High-performance steel welded I-sections in bending are classified as compact, non-compact and slender sections in both the American specifications (AISC, 2013, 2016) and Australian standard (AS, 1998). The American and Australian standards adopt M_{pl} as the design bending moment resistances for compact welded I-section beams and take into account partial plasticity in the predictions of cross-section bending moment resistances for non-compact welded I-section beams, while for slender welded I-section beams, the effective bending moment capacities M_{eff} are taken as the design bending moment resistances. Regarding the cross-section compression resistances, all the considered design standards prescribe the use of the yield loads, given as $A\sigma_{0.2}$ or Af_y , for non-slender (Class 1, 2 and 3) welded I-sections and effective compression capacities N_{eff} for slender (Class 4) welded I-sections. The effective cross-section bending and compression capacities for slender sections are calculated based on the effective width method, originally proposed by Winter (1947) and later modified in each design standard, which treats the cross-section as an assemblage of isolated plate elements without accounting for element interaction and considers loss of effectiveness due to local buckling through a reduction in plate element width.

Regarding high-performance steel welded I-sections under combined compression and bending, the European codes EN 1993-1-4 (CEN, 2015) and EN 1993-1-12 (CEN, 2007) adopt nonlinear design interaction formulae, as derived based on full plasticity throughout the whole cross-section, for Class 1 and 2 welded I-sections. A linear design interaction formula is employed in the European codes (CEN, 2007, 2015) for Class 3 and Class 4 welded I-sections under combined loading, which is a simple linear summation of the utilisation ratio for each stress resultant, with an upper limit equal to unity. The American specifications (AISC, 2013, 2016) adopt a bi-linear design interaction formula for high-performance steel welded I-sections under combined loading, while the Australian standard (AS, 1998) uses a linear design interaction formula. It is worth noting that the AS linear interaction formula is the same as its EC3 counterpart, with the only difference lying in the calculation of the cross-section bending moment resistances.

Detailed discussions on the codified local buckling design rules for high-performance steel welded I-sections under isolated and combined loadings are given in Chapters 3–5.

2.4.2.3 Continuous strength method (CSM)

The continuous strength method, originally proposed by Gardner (2002) and Gardner and Nethercot (2004a), further developed by Gardner and Ashraf (2006), Ashraf et al. (2006) and Gardner (2008b) and finalised by Afshan and Gardner (2013), Zhao et al. (2017) and Zhao and Gardner (2018), is a deformation-based design method that enables a rational exploitation of material strain hardening in the calculation of cross-section resistances. The main characteristics of the CSM (Afshan & Gardner, 2013; Zhao et al., 2017; Zhao & Gardner, 2018) lie in (i) the employment of a continuous ‘base curve’ to determine the maximum attainable strain for the considered cross-section under the applied loading and (ii) the adoption of an elastic, linear hardening material model to enable design stresses greater than the 0.2% proof stress to be

attained. The CSM replaces the codified cross-section classification framework by a continuous treatment and accounts for the favourable material strain hardening, thus leading to a high level of accuracy and consistency in the predictions of cross-section resistances. Detailed discussions on the CSM for high-performance steel welded I-sections are provided in Chapters 3 and 4.

2.4.3 Member stability

2.4.3.1 Previous experimental investigations

A series of testing programmes were previously conducted to investigate the global buckling behaviour and resistances of high-performance steel welded I-section members subjected to compression (i.e. columns) and combined compression and bending (i.e. beam-columns). Table 2.6 and Table 2.7 summarise the previous column tests and beam-column tests on high-performance steel welded I-sections, respectively.

2.4.3.2 Codified design rules for global instability

The current European codes (CEN, 2007, 2015), American specifications (AISC, 2013, 2016) and Australian standard (AS, 2016) adopt the same concept of using buckling curves for the design of high-performance steel welded I-section columns failing by flexural buckling. Regarding high-performance steel welded I-section columns susceptible to local–flexural interactive buckling, the current design standards (CEN, 2007, 2015; AISC, 2013, 2016; AS, 2016) combine the use of effective width method for considering local buckling and buckling curves for accounting for global buckling. Detailed discussions on the codified column buckling design rules are provided in Chapters 6 and 7. With regard to the design of high-performance steel welded I-section beam-columns under combined compression and bending moment, the existing design standards (CEN, 2007, 2015; AISC, 2013, 2016;

AS, 2016) employ interaction curves, with the end points being member resistances under the isolated loading conditions and the shapes defined by interaction factors. Specifically, the current European code EN 1993-1-12 (CEN, 2007) for high strength steels provide no specific design rules for beam-columns, but directly adopts the two alternative design interaction formulae given in EN 1993-1-1 (CEN, 2014) for carbon steel beam-columns, which are referred to as Method 1 and Method 2 (Greiner, 2001). Four interaction factors (k_{yy} , k_{yz} , k_{zy} and k_{zz}) are used to consider all the plasticity effects and second-order geometric effects in the interaction between compressive force and bending moment, which were derived based on numerical simulations of beam-columns with uniform first-order bending moment along the member length. The interaction factors k_{yy} and k_{zz} allow for the interaction effects when the bending plane is the same as the buckling plane, while the interaction factors k_{yz} and k_{zy} concern the plane perpendicular to the plane of buckling. Previous studies indicated that the interaction factors depend on the member slenderness, compressive force and cross-section shape. Detailed derivation of the interaction factors used in Method 1 and Method 2 were reported by Boissonnade et al. (2002, 2004, 2006) and Lindner (1997, 2003), Rusch and Lindner (2004) and Greiner and Lindner (2006), respectively. With regard to the design of stainless steel beam-columns, EN 1993-1-4 (CEN, 2015) employs the Method 2 interaction formulae, but with modified interaction factors to consider the specific effect of nonlinear material response and high degree of strain hardening of stainless steel on member instability. The design interaction formulae for high-performance steel cross-sections under combined loading, as given in the current American and Australian standards (AISC, 2013, 2016; AS, 2016), are also used for beam-columns, but with the use of amplification factors to take into consideration the second-order effects. Detailed discussions on the codified beam-column interaction formulae are presented in Chapter 8.

Table 2.6: Summary of previous column tests on high-performance steel welded I-sections

Steel	Type and grade	Reference	No. of tests
Stainless steel	Austenitic EN 1.4301	Burgan et al. (2000)	12
		Yuan et al. (2015)	5
		Yang et al. (2016)	12
	Austenitic EN 1.4307	Gardner et al. (2016)	11
	Austenitic EN 1.4404	Gardner et al. (2016)	3
	Austenitic EN 1.4571	Gardner et al. (2016)	8
	Duplex EN 1.4462	Yuan et al. (2015)	5
		Yang et al. (2016)	12
	Ferritic EN 1.4512	Bredenkamp and Van den Berg (1995)	13
High strength steel	S460	Ban et al. (2012)	7
		Wang et al. (2012b)	6
		Zhou et al. (2013)	6
	S690	Rasmussen and Hancock (1995)	5
		Shi et al. (2012)	4
		Li et al. (2016)	6
		Ma et al. (2018b)	7
	S960	Shi et al. (2012)	4
		Ban et al. (2013)	6
		Su et al. (2021b)	10

Table 2.7: Summary of previous beam-column tests on high-performance steel welded I-sections

Steel	Type and grade	Reference	No. of tests
Stainless steel	Austenitic EN 1.4301	Burgan et al. (2000)	8
		Zheng et al. (2015)	5
		Bu and Gardner (2019)	6
		Yang et al. (2019)	10
		Kucukler et al. (2020)	5
	Austenitic EN 1.4571	Bu and Gardner (2019)	6
	Duplex EN 1.4462	Yang et al. (2019)	10
High strength steel	S460	Li et al. (2012)	6
	S690	Ma et al. (2017)	8

2.5 Numerical modelling

In parallel with laboratory experiments, finite element (FE) analyses have been widely used for numerically simulating the structural behaviour of thin-walled steel members. The general-purpose FE analysis software ABAQUS (2014) is employed for the numerical modelling programmes presented in this thesis. In-depth research on the numerical modelling of high-performance steel welded I-section members has been carried out previously; the key modelling assumptions and techniques relevant to the FE analyses conducted in this thesis are discussed herein.

2.5.1 Element type

Shell elements are customarily used to simulate thin-walled steel components, which result in better computational efficiency over solid elements, but still provide very good numerical accuracy. The ABAQUS element library (2014) contains a series of shell elements. The shell element adopted in the numerical modelling reported in this thesis is S4R (ABAQUS, 2014), a four-noded doubly curved shell element with reduced integration and finite membrane strains, which has been shown to perform well in previous numerical modelling of stainless steel (Saliba & Gardner, 2013; Yuan et al., 2015; Bu & Gardner, 2018) and high strength steel (Ricles et al., 1998; Ma et al., 2018a) welded I-section members.

2.5.2 Modelling of material response and membrane residual stresses

ABAQUS (2014) requires the inputted material properties to be specified in the format of true stress and true plastic strain. In this thesis, the engineering stress–strain curves measured from tensile coupon tests were firstly converted into the true stress–plastic strain curves, according to Equations (2.12) and (2.13), where σ_{true} is the true stress, ϵ_{ln}^{pl} is the true plastic strain, σ_{nom} is the engineering stress and ϵ_{nom} is the

engineering strain, and then assigned to FE models. The welding-induced membrane residual stresses, derived from the predictive models proposed in this thesis, were incorporated into FE models through the ‘*INITIAL CONDITIONS’ command (Saliba & Gardner, 2013; ABAQUS, 2014; Yuan et al., 2015; Bu & Gardner, 2018).

$$\sigma_{true} = \sigma_{nom} (1 + \varepsilon_{nom}) \quad (2.7)$$

$$\varepsilon_{ln}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (2.8)$$

2.5.3 Incorporation of initial geometric imperfections

Initial local and global geometric imperfections were incorporated into the developed finite element models in the form of the respective lowest elastic buckling mode shapes under the applied loading (Yuan et al., 2015; Zhao et al., 2016c; Buchanan et al., 2018); these shapes generally represent the most unfavourable imperfection patterns and can be derived through a prior elastic eigenvalue buckling analysis (ABAQUS, 2014). In the numerical modelling presented in this thesis, various local and global imperfection magnitudes have been adopted to scale the corresponding geometric imperfection patterns, for the purpose of assessing the sensitivity of FE models to imperfection magnitudes and seeking the most appropriate imperfection magnitudes to be used in the parametric studies.

2.5.4 Analysis technique

The structural behaviour of thin-walled steel members is affected by material and geometric nonlinearities. Upon development of FE models, the modified Riks method (ABAQUS, 2014) was employed to perform geometrically and materially nonlinear analyses to derive the numerical failure loads, full load–deformation responses and failure modes.

2.6 Concluding remarks

This chapter has provided an overview of the previous research work on high-performance steel welded I-section structural components and the recent developments in design guidance and numerical modelling. The literature review generally indicates that (i) the structural behaviour of S690 high strength steel welded I-section components remains relatively unexplored and (ii) although comprehensive research on stainless steel welded I-section components has been conducted previously, the structural performance of welded I-section components made of the new grade EN 1.4420 high-chromium stainless steel has not been investigated and still requires verification. Therefore, investigations of welded I-section structural components made of grade EN 1.4420 high-chromium stainless steel and grade S690 high strength steel and development of efficient design rules are the objectives of the present thesis.

CHAPTER 3

CROSS-SECTION BEHAVIOUR OF HIGH-CHROMIUM STAINLESS STEEL WELDED I-SECTIONS

3.1 Introduction

This chapter presents a thorough experimental and numerical study into the material responses and local stability of welded I-sections made of the new grade EN 1.4420 high-chromium austenitic stainless steel. A testing programme was firstly conducted on four gas metal arc welded I-sections, and included material tensile coupon tests, membrane residual stress measurements, initial local geometric imperfection measurements, eight stub column tests and eight in-plane four-point bending tests, with four for each principal axis. A parallel numerical modelling programme was then carried out, where finite element models were developed and validated against the obtained test results and afterwards used to perform parametric studies to generate further numerical data over a wide range of cross-section aspect ratios and dimensions. The obtained test and numerical results were adopted to evaluate the applicability of the codified slenderness limits and local buckling design rules given in both Eurocode EN 1993-1-4 (CEN, 2015) and American design guide AISC 27 (AISC, 2013) to the new high-chromium stainless steel welded I-sections. The accuracy of the continuous strength method (Afshan & Gardner, 2013; Zhao et al., 2017) for the design of the new high-chromium stainless steel welded I-sections was also evaluated. The findings of this research work have been reported in Sun and Zhao (2019).

3.2 Testing

3.2.1 General

A comprehensive testing programme was performed to investigate the material responses and local stability of welded I-sections made of the new grade EN 1.4420 high-chromium stainless steel. Four different I-section sizes were adopted in the testing programme: I-200×100×5, I-150×150×5, I-150×120×6 and I-150×100×6, which were fabricated by gas metal arc welding (GMAW) from 5 mm and 6 mm thick plates. The detailed welding process is described as follows. Three stainless steel plates were positioned on the flat work bench to form a well-aligned I-shaped profile. GMAW was then started at the same time from different sides of the web-to-flange junctions at different ends. This welding strategy helped minimise the member distortion during welding, with the welding quality satisfying the requirements specified in EN ISO 3834-2 (CEN, 2005). It is worth noting that the commonly used stainless steel electrode A309, with ultimate stress equal to 620 MPa, was used for welding. The welding velocity was equal to 300 mm/min, with a welding current of 125 A and arc voltage of 20 V. The adopted identifier for each specimen comprises the section size, a letter ('A' or 'B') and a number ('1' or '2'). Specifically, the letter distinguishes the test type, where A identifies a stub column and B indicates a beam, while the number differentiates the normally identical specimens. Specimen I-200×100×5-A1, for instance, represents an I-section stub column, the dimensions of which are 200 mm (outer section height h), 100 mm (flange width b_f) and 5 mm (plate thickness t). The overall testing programme included material testing, membrane residual stress measurements, initial local geometric imperfection measurements, eight stub column tests and eight four-point bending tests, with four for each principal axis.

3.2.2 Material testing

Material tensile coupon tests were performed to obtain the material stress–strain responses of the studied grade EN 1.4420 high-chromium austenitic stainless steel.

For each plate thickness, two tensile coupons, with the geometric dimensions conforming to the requirements given in EN ISO 6892-1 (CEN, 2016b), were tested using an INSTRON 250 kN servo-hydraulic tensile testing machine under displacement control. The initial loading rate was set to be equal to 0.05 mm/min up to the nominal 0.2% proof stress $\sigma_{0.2}$, beyond which an increased rate of 0.80 mm/min was employed until the fracture of the coupon. The adopted loading rates were in compliance with the requirements specified in EN ISO 6892-1 (CEN, 2016b). Figure 3.1 depicts the tensile coupon test setup, where an extensometer with 50 mm gauge length is mounted onto the middle part of the coupon and two strain gauges are attached to both sides of the coupon at mid-height. During the material testing, a data acquisition system Bluehill was used to record the tensile force, displacement and strains from the load cell, extensometer and strain gauges at 1 second intervals, respectively. The measured data from the first stage with the lower loading rate were used to determine the initial Young's modulus E and the 0.2% proof stress $\sigma_{0.2}$, while the 1.0% proof stress $\sigma_{1.0}$, the ultimate stress σ_u and the strains at the ultimate stress and fracture ε_u and ε_f were determined based on the data measured from the second stage with the higher loading rate. The measured stress–strain curves for 5mm and 6mm thick plates are plotted in Figures 3.2(a) and 3.2(b), respectively, and the key average measured material properties are reported in Table 3.1.

Figure 3.1: Material tensile coupon test setup

(a) 5mm thick plate

(b) 6mm thick plate

Figure 3.2: Measured stress-strain curves for new grade EN 1.4420 high-chromium stainless steel

Table 3.1: Average measured tensile material properties of new grade EN 1.4420 high-chromium stainless steel

Plate thickness	E (MPa)	$\sigma_{0.2}$ (MPa)	$\sigma_{1.0}$ (MPa)	σ_u (MPa)	ε_u (%)	ε_f (%)	R–O coefficients	
							n	$m_{1.0}$
5 mm	187800	326	379	688	46	67	3.2	2.9
6 mm	187900	339	398	718	45	69	3.5	2.5

3.2.3 Membrane residual stress measurements

Welded steel sections generally contain high levels of membrane residual stresses, which are introduced during the welding process and can lead to premature failure of steel structural components; therefore, the membrane residual stress distributions and amplitudes in high-chromium stainless steel welded I-sections were measured herein, based on the sectioning method and in conformity with the procedures recommended in Ziemian (2010). A total of four sets of membrane residual stress measurements were conducted on I-200×100×5 and I-150×100×6, with two repeated measurements for each I-section size. Figures 3.3 and 3.4 depict the locations and dimensions of the strips cut for membrane residual stress measurements; the nominal length of each strip is 150 mm, with the width being either 7 mm or 10 mm (depending on the overall width the plate element of the I-section). Prior to sectioning, two gauge holes with a diameter of 1.98 mm were drilled along the centreline of the exterior face of each strip and at a distance of 25 mm from the strip ends, through the use of an automatic dot puncher, leading to the nominal length between the gauge holes L_0 equal to 100 mm; the actual strip length between the two gauge holes was thereafter measured using a Demec gauge with 100 mm gauge length. Upon length measurements of strips within the I-sections, each specimen was cut into strips, allowing for the release of membrane residual stress; a typical sectioned I-150×100×6 specimen is shown in Figure 3.5. The Demec gauge was used again to measure the lengths of strips between the gauge holes after sectioning. For each strip, the readings of the Demec gauge

taken before and after sectioning are denoted as r_1 and r_2 , respectively. In order to eliminate the influence of temperature changes during the membrane residual stress measurements, a temperature reference bar, made of the same material as the welded I-section specimens, was used, with the lengths measured before and after sectioning denoted as t_1 and t_2 , respectively.

Figure 3.3: Locations of strips cut for residual stress measurements (dimensions in mm)

Figure 3.4: Locations and dimensions of strips within I-sections (dimensions in mm)

Figure 3.5: Typical sectioned I-150×100×6 specimen

On the basis of the readings taken from the Demec gauge and temperature reference bar before and after sectioning, the strain ε_0 induced due to the release of membrane residual stress can be determined from Equation (3.1); note that positive relieved strains calculated from Equation (3.1) signify tensile residual stresses, while negative relieved strains indicate compressive residual stresses. For strips in the vicinity of welds, where high through-thickness residual stress gradients exist and lead to slightly curved strips upon cutting, the relieved strains determined from Equation (3.1) were further corrected, based on Equation (3.2) (Tebedge et al., 1973), to eliminate the effect of longitudinal curvature, where ε_c is the corrected relieved strain, and δ/L_0 is defined as the ratio of the maximum deviation from a straight reference line connecting the two gauge holes of the strip δ to the nominal length between the gauge holes L_0 .

$$\varepsilon_0 = \frac{(r_1 - t_1) - (r_2 - t_2)}{L_0 + r_1 - t_1} \quad (3.1)$$

$$\varepsilon_c = \varepsilon_0 + \frac{(\delta / L_0)^2}{6(\delta / L_0)^4 + 1} \quad (3.2)$$

Upon determination of the relieved strains for all the strips, the corresponding membrane residual stresses were then back-calculated through multiplying the relieved strains by the Young's modulus obtained from the tensile coupon tests, and are shown in Figures 3.6(a) and 3.6(b) for the two considered welded I-sections I-200×100×5 and I-150×100×6, respectively, with positive and negative values corresponding to tensile and compressive residual stresses, respectively. The measured data points in Figures 3.6(a) and 3.6(b) generally show a good agreement between the two repeated membrane residual stress measurements for each I-section size. The obtained peak tensile and compressive residual stresses are normalised by the corresponding measured 0.2% proof stresses, and then reported in Table 3.2, where the subscripts 'w' and 'f' represent web and flange, respectively, while the subscripts 't' and 'c' denote tensile and compressive residual stresses, respectively. Figures 3.7(a) and 3.7(b) present the measured membrane residual stress patterns and amplitudes in a normalised format, together with the codified predictive models given in the Swedish regulations BSK 99 (2003) and European convention ECCS (1976) for carbon steel welded I-sections and a recently proposed predictive model in Yuan et al. (2014a) for stainless steel welded I-sections. Note that the normalised position at the origin point 0.0 corresponds to the web-to-flange junction, while the coordinate of the normalised position at 1.0 represents the mid-point of the web or the tip of the flange. The considered predictive models for membrane residual stresses in welded I-sections follow the same general pattern, as shown in Figure 3.8, but with different distribution parameters (a , b , c and d) and peak tensile and compressive residual stress magnitudes, as given in Table 3.3. It is evident in Figures 3.7(a) and 3.7(b) that the two codified predictive models, originally developed for carbon steel welded I-sections, generally over-estimate the peak tensile residual stresses but lead to rather conservative peak compressive residual stress predictions for the new high-chromium stainless steel welded I-sections; this can be principally attributed to the distinct nonlinear material behaviour of stainless steel in comparison with the linear material

response of carbon steel. Moreover, it was also found that the residual stress transition regions existed in stainless steel welded I-sections are substantially wider than those defined by the codified predictive models for carbon steel welded I-sections. The recently proposed predictive model in Yuan et al. (2014a) for normal austenitic stainless steel welded I-sections was generally shown to yield a good representation of membrane residual stress distributions in their high-chromium counterparts, though with conservative peak tensile residual stress predictions and unsafe peak compressive residual stress predictions. Revised predictive model for membrane residual stresses in high-chromium stainless steel welded I-sections was then proposed, with the new distribution parameters and peak residual stress magnitudes given in Table 3.3, and are also plotted in Figure 3.7, indicating excellent agreement with the measured membrane residual stress data points.

Figure 3.6: Measured membrane residual stress patterns and amplitudes

(a) Outstand flange

(b) Internal web

Figure 3.7: Comparisons between measured membrane residual stresses and predictive models

Table 3.2: Summary of measured peak membrane residual stresses of high-chromium stainless steel welded I-sections

Specimen	$\sigma_{fi}/\sigma_{0.2}$	$\sigma_{wt}/\sigma_{0.2}$	$\sigma_{fc}/\sigma_{0.2}$	$\sigma_{wc}/\sigma_{0.2}$
I-200×100×5-1	0.67	0.57	-0.62	-0.41
	0.70	0.58	-0.63	
I-200×100×5-2	0.61	0.49	-0.51	-0.33
	0.56	0.47	-0.48	
I-150×100×6-1	0.59	0.47	-0.43	-0.50
	0.67	0.46	-0.41	
I-150×100×6-2	0.53	0.45	-0.53	-0.58
	0.64	0.50	-0.50	
Mean	0.62	0.50	-0.51	-0.46
Maximum	0.70	0.58	-0.63	-0.58

Figure 3.8: General membrane residual stress pattern for welded I-sections

Table 3.3: Membrane residual stress predictive models for stainless steel welded I-sections

Residual stress predictive model	$\sigma_{ft} = \sigma_{wt}$	$\sigma_{fc} = \sigma_{wc}$	a	b	c	d
BSK 99 (2003)	$1.0\sigma_{0.2}$	From equilibrium	$0.75t_f$	$1.5t_f$	$1.5t_w$	$1.5t_w$
ECCS (1976)	$1.0\sigma_{0.2}$	$0.25 \sigma_{0.2}$	$0.05b_f$	$0.15b_f$	$0.075h_w$	$0.05h_w$
Yuan et al. (2014a)	$0.8\sigma_{0.2}$	From equilibrium	$0.225b_f$	$0.05b_f$	$0.025h_w$	$0.225h_w$
Proposed model	$0.8\sigma_{0.2}$	From equilibrium	$0.2b_f$	$0.15b_f$	$0.075h_w$	$0.2h_w$

Note: h_w is the clear distance between the flanges, t_w is the web thickness and t_f is the flange thickness.

3.2.4 Initial local geometric imperfection measurements

Initial local geometric imperfections can greatly affect the local buckling behaviour of thin-walled stainless steel open and tubular section structural components, e.g., leading to low load-carrying capacities and steep post-ultimate responses, and were thus measured for all the high-chromium stainless steel welded I-section stub column and beam specimens, based on the test setup and procedures recommended in Schafer and Peköz (1998). Figure 3.9 shows the test setup for initial local geometric imperfection measurements, where a linear variable displacement transducer (LVDT) is attached to the head of a milling machine, with specimens clamped tightly in a vise on the moving machine bed. For each plate element of the welded I-section, three imperfection measurements were performed along the centreline and two edge lines, and over the central 300 mm of each stub column and central 600 mm of each beam, in order to eliminate the end flaring effect (Cruise & Gardner, 2006). The initial local geometric imperfections for each plate element of the welded I-section specimen were taken as the deviations from a linear regression surface fitted to the data set. For each specimen, the measured maximum initial local imperfection magnitudes of the internal web and two outstand flanges (ω_w , ω_{f1} and ω_{f2}) are reported in Table 3.4, while the initial local geometric imperfection magnitude of the specimen ω_0 was taken as the maximum of ω_w , ω_{f1} and ω_{f2} .

Figure 3.9: Setup for initial local geometric imperfection measurements

3.2.5 Stub column tests

For each considered high-chromium stainless steel welded I-section, two repeated stub column tests were conducted to investigate the cross-sectional resistances and deformation capacities under compression. The nominal stub column length was selected in accordance with the recommendations given in Ziemian (2010), and set to be equal to the three times the outer section height herein; the selected specimen lengths are short enough to prevent the occurrence of member global buckling, but still sufficiently long to contain representative membrane residual stress distributions and initial local geometric imperfection patterns. The two ends of each stub column specimen were firstly milled flat and then fully deburred, ensuring a uniform compressive stress distribution in the specimen during the testing. Table 3.4 reports the measured geometric properties for each stub column specimen, where L is the member length, h is the outer section height, b_f is the flange width and t is the plate thickness – see Figure 3.10.

Figure 3.11: Notations of welded I-section

The stub column test setup is shown in Figure 3.11, where two LVDTs are positioned at the specimen ends to record the end shortening and two strain gauges are attached to the web-to-flange junctions of the specimen at mid-height to measure the longitudinal compressive strains. All the stub column tests were conducted using a displacement-controlled INSTRON 2000 kN servo-hydraulic testing machine at a loading rate of 0.3 mm/min. As highlighted in previous experimental studies on stub columns (Centre for Advanced Structural Engineering, 1990; Gardner & Nethercot, 2004a), the LVDT readings contain both the end shortenings of the stub column specimens and the deformation of the testing machine itself. The end shortening values obtained from the LVDTs were then corrected based on the readings of the strain gauges, following the procedures described in (Centre for Advanced Structural Engineering, 1990). The corrected load–end shortening curves for all the tested high-chromium stainless steel welded I-section stub column specimens are depicted in Figure 3.12, while the key test results are reported in Table 3.5, where N_u is the failure load, δ_u is the end shortening at failure and $N_u/(A\sigma_{0.2})$ is the ratio of the failure load to cross-section yield load, where A is the gross cross-section area of the welded I-section. All the eight tested stub column specimens failed by local buckling, with a typical failed specimen I-150×100×6-A1 shown in Figure 3.13.

Figure 3.11: Stub column test setup

Figure 3.12: Load–end shortening curves of high-chromium stainless steel welded I-section stub column specimens

Table 3.4: Measured geometric dimensions and initial local geometric imperfections of high-chromium stainless steel welded I-section stub column and beam specimens

Specimen ID	L (mm)	h (mm)	b_f (mm)	t (mm)	ω_w (mm)	ω_{f1} (mm)	ω_{f2} (mm)	ω_0 (mm)
I-200×100×5-A1	600.2	199.74	99.27	5.02	0.17	0.33	0.25	0.33
I-200×100×5-A2	600.5	199.62	99.28	5.02	0.14	0.44	0.52	0.52
I-150×150×5-A1	448.0	149.57	149.09	5.09	0.13	0.23	0.23	0.23
I-150×150×5-A2	447.5	149.64	149.15	5.07	0.06	0.13	0.16	0.16
I-150×120×6-A1	450.0	149.29	119.33	6.01	0.09	0.10	0.10	0.10
I-150×120×6-A2	448.4	149.45	119.38	6.00	0.07	0.12	0.10	0.12
I-150×100×6-A1	448.3	149.35	99.44	6.13	0.04	0.15	0.11	0.15
I-150×100×6-A2	448.1	149.31	99.43	6.08	0.06	0.14	0.14	0.14
I-200×100×5-B1	1997	199.48	99.32	5.01	0.15	0.28	0.22	0.28
I-200×100×5-B2	1998	199.20	99.01	4.99	0.15	0.21	0.18	0.21
I-150×150×5-B1	2003	149.97	149.06	5.09	0.11	0.42	0.28	0.42
I-150×150×5-B2	2000	150.30	148.86	5.15	0.17	0.40	0.50	0.50
I-150×120×6-B1	2000	149.49	119.27	5.95	0.22	0.33	0.25	0.33
I-150×120×6-B2	2000	148.96	119.54	6.08	0.09	0.42	0.44	0.44
I-150×100×6-B1	1998	149.67	99.26	6.10	0.13	0.33	0.36	0.36
I-150×100×6-B2	1999	149.22	99.54	6.10	0.11	0.24	0.34	0.34

Table 3.5: Key test results for high-chromium stainless steel welded I-section stub column specimens

Specimen ID	N_u (kN)	δ_u (mm)	$N_u/(A\sigma_{0.2})$
I-200×100×5-A1	605.9	3.16	0.95
I-200×100×5-A2	585.3	3.50	0.92
I-150×150×5-A1	687.3	1.79	0.95
I-150×150×5-A2	690.3	1.90	0.95
I-150×120×6-A1	851.1	4.27	1.15
I-150×120×6-A2	843.3	4.13	1.14
I-150×100×6-A1	785.3	5.80	1.18
I-150×100×6-A2	778.4	5.39	1.17

Figure 3.13: Test and FE failure modes for stub column specimen I-150×100×6-A1

3.2.6 Four-point bending tests

For each of the four adopted high-chromium stainless steel welded I-sections, two in-plane four-point bending tests were conducted about both the major and minor principal axes, to study the cross-sectional flexural behaviour and resistances. Table 3.4 reports the measured geometric dimensions of the eight beam specimens. The test setup for beams in four-point bending about the minor principal axis is depicted in Figure 3.14, where two pairs of steel rollers are adopted to provide the four-point bending configuration, with one pair located at a distance of 100 mm from the end

sections of the beam specimen and the other pair placed at the third-points of the flexural span (i.e. the span between the two end rollers) of the beam specimen, solid wooden blocks are inserted into the I-section beam at both the supports and loading points and stiffening plates are also clamped at these positions, to eliminate the possibility of local bearing failure, and three string transducers are positioned at mid-span and two loading points to capture the respective vertical deflections. Similar test setup was also employed for the four-point bending tests about the cross-section major principal axis, as shown in Figure 3.15, but with additional lateral restraints, provided by a series of G-clamps, to prevent any lateral and torsional deformation and thus lateral-torsional buckling. The four-point bending tests were conducted at a loading rate of 1.5 mm/min.

Table 3.6 reports the key test results for the high-chromium stainless steel welded I-section beam specimens, where M_u is the failure moment, M_u/M_{pl} and M_u/M_{el} are the ratios of the failure moment to the cross-section plastic and elastic moment capacities, respectively, where M_{pl} and M_{el} are calculated as the cross-section plastic (W_{pl}) and elastic (W_{el}) moduli multiplied by the material 0.2% proof stress $\sigma_{0.2}$, respectively, and R is the rotation capacity, as given by Equation (3.3), in which $\kappa_{pl}=M_{pl}/(EI)$, where I is the second moment of area about the considered bending axis, and κ_u is the curvature where the descending part of the moment–curvature curve drops to M_{pl} . The normalised moment–curvature curves of the tested beams are displayed in Figure 3.16, where the curvature κ is determined from Equation (3.4) (Chan & Gardner, 2008), where D_L and D_M are the vertical deflections at the loading points and mid-span, respectively, and L_m is the distance between the loading points. All of the tested beam specimens exhibited in-plane local buckling failure modes, with typical failure modes of specimens I-150×150×5-B1 and I-150×150×5-B2 subjected to major-axis and minor-axis bending depicted in Figures 3.17 and 3.18, respectively.

$$R = \frac{\kappa_u}{\kappa_{pl}} - 1 \quad (3.3)$$

$$\kappa = \frac{8(D_M - D_L)}{4(D_M - D_L)^2 + L_m^2} \quad (3.4)$$

Figure 3.14: Minor-axis four-point bending test setup

Figure 3.15: Major-axis four-point bending test setup

Table 3.6: Key test results for high-chromium stainless steel welded I-section beam specimens

Specimen ID	Axis of bending	M_u (kNm)	M_u/M_{pl}	M_u/M_{el}	R
I-200×100×5-B1	Major axis	50.6	1.10	1.27	>2.62
I-150×150×5-B1		45.3	1.03	1.14	2.66
I-150×120×6-B1		46.8	1.10	1.24	5.22
I-150×100×6-B1		43.6	1.15	1.32	>3.44
I-200×100×5-B2	Minor axis	9.5	1.13	1.76	>7.05
I-150×150×5-B2		19.0	1.02	1.55	4.05
I-150×120×6-B2		17.6	1.21	1.87	>5.90
I-150×100×6-B2		12.6	1.22	1.91	>4.69

Figure 3.16: Normalised moment–curvature curves of high-chromium stainless steel welded I-section beam specimens

Figure 3.17: Test and FE failure modes for beam specimen
I-150×150×5-B1 in major-axis bending

Figure 3.18: Test and FE failure modes for beam specimen
I-150×150×5-B2 in minor-axis bending

3.3 Numerical modelling

3.3.1 General

Following the testing programme, a numerical modelling programme was performed, using the finite element analysis software ABAQUS (2014). The FE models were firstly developed and validated against the test results, and then adopted to perform parametric studies to generate further numerical data over a wide range of cross-section aspect ratios and dimensions.

3.3.2 Development of finite element models

The shell element S4R (ABAQUS, 2014) has been widely and successfully adopted in previous nonlinear numerical modelling of stainless steel welded I-section components (Saliba & Gardner, 2013; Yuan et al., 2015; Bu & Gardner, 2018), and was thus also adopted in the present finite element modelling. The shell element size was determined through a prior mesh sensitivity study, where a series of element sizes ranging from $0.5t$ to $5t$ were examined. It was generally found that the element size equal to the plate thickness t led to an accurate incorporation of the membrane residual stresses into the FE models, while still offering computational efficiency, and was thus adopted herein. With regard to the material modelling of the new grade EN 1.4420 high-chromium stainless steel, the (measured) engineering stress–strain responses were converted into the true stress–strain responses, and then assigned to the stub column and beam FE models. The membrane residual stresses in each high-chromium stainless steel welded I-section stub column and beam specimen, as derived from the predictive model proposed in Section 3.2.3, were incorporated into the corresponding FE model by means of the ‘*INITIAL CONDITIONS’ command (ABAQUS, 2014); a typical membrane residual stress distribution pattern incorporated in the FE models for the I-150×150×5 stub column specimens is shown in Figure 3.19. Suitable boundary conditions were then applied to the FE models to replicate the experimental boundary conditions. Specifically, for each stub column FE model, its two end sections were coupled to two concentric reference points, with

one allowed for longitudinal translation and the other one restrained against all degrees of freedom, to mimic the fixed-ended boundary conditions adopted in the stub column tests. For each beam FE model, the end sections at the supports were coupled to two eccentric reference points, located at the bottom web-to-flange junctions and allowing translation along the longitudinal direction and rotation about the bending axis at one end and only rotation about the same bending axis at the other end, to achieve the same pin-ended boundary conditions employed in the four-point bending tests; moreover, the cross-sections at the two loading points were set as rigid bodies, only allowing vertical and longitudinal translations plus rotation about the bending axis. Regarding the modelling of major-axis four-point bending tests, continuous restraints against lateral deflections and torsional rotations were also applied to the beam FE models at the same cross-sections as those restrained by G-clamps in the tests, to avoid the occurrence of lateral-torsional buckling.

Figure 3.19: Typical membrane residual stress distribution and amplitudes (in MPa) in modelled I-section I-150×150×5 (Positive values indicate tensile residual stresses while negative values indicate compressive residual stresses)

The structural behaviour of thin-walled section steel components can be greatly influenced by their initial geometric imperfections; therefore, incorporation of the initial local geometric imperfections into the FE models was deemed necessary, in order to accurately simulate the test responses of high-chromium stainless steel

welded I-section stub columns and beams. The initial local geometric imperfection patterns were taken as the lowest elastic buckling mode shapes subjected to the applied loading, derived from an elastic eigenvalue buckling analysis (Saliba & Gardner, 2013; Zhao et al., 2016b; Liang et al., 2019). Three initial local imperfection magnitudes, including (i) the measured imperfection magnitude ω_0 , as reported in Table 3.4, (ii) 1/10 of the plate thickness t and (iii) the predicted value from the modified Dawson and Walker model $\omega_{D\&W}$ (Dawson & Walker, 1972; Gardner & Nethercot, 2004b), were adopted to scale the imperfection patterns, in order to evaluate the sensitivity of the developed FE models to initial local geometric imperfections and seek the most appropriate local geometric imperfection magnitude to be adopted in the parametric studies. Following the successful incorporation of the initial local geometric imperfections into the developed FE models, static Riks analysis (ABAQUS, 2014), accounting for both material and geometric nonlinearities, was performed to derive the numerical failure loads and moments, load–deformation curves and failure modes.

3.3.3 Validation of finite element models

In this section, assessment of the accuracy of the developed FE models was carried out, based on a series of comparisons between the test and numerical results, including the failure loads and moments, load–deformation curves and failure modes. Tables 3.7 and 3.8 report the numerical to test failure load and moment ratios for the stub column and beam specimens, respectively. It was generally found that all the three initial local geometric imperfection magnitudes result in accurate and consistent predictions of the test failure loads and moments, with the best agreement achieved by adopting the measured initial local imperfection magnitude ω_0 . A comparison between the test and FE load–end shortening curve for a typical stub column specimen I-150×150×5-A2 is depicted in Figure 3.20, while the test and FE moment–curvature curves for a typical beam specimen I-150×150×5-B2 are compared in Figure 3.21, both indicating that the test load–deformation curves are well captured by the corresponding numerical curves. The numerical local buckling failure modes were also found to display excellent agreement with their experimental counterparts,

as shown in Figures 3.14, 3.17 and 3.18. Overall, the developed FE models have been proven to be capable of accurately replicating the test responses of high-chromium stainless steel welded I-section stub columns and beams, and thus deemed to be validated.

Table 3.7: Comparison of FE failure loads with test failure loads for varying initial local geometric imperfection magnitudes

Specimen ID	Finite element N_u / Test N_u		
	ω_0	$t/10$	$\omega_{D\&W}$
I-200×100×5-A1	0.97	0.96	1.03
I-200×100×5-A2	0.99	1.00	1.04
I-150×150×5-A1	0.98	0.97	1.03
I-150×150×5-A2	1.00	0.96	1.02
I-150×120×6-A1	0.98	0.94	1.00
I-150×120×6-A2	0.98	0.94	1.01
I-150×100×6-A1	0.98	0.95	1.03
I-150×100×6-A2	0.99	0.95	1.04
Mean	0.98	0.96	1.02
COV	0.01	0.02	0.01

Table 3.8: Comparison of FE failure moments with test failure moments for varying initial local geometric imperfection magnitudes

Specimen ID	Finite element M_u / Test M_u		
	ω_0	$t/10$	$\omega_{D\&W}$
I-200×100×5-B1	0.98	0.96	1.01
I-200×100×5-B2	0.96	0.95	0.99
I-150×150×5-B1	0.94	0.92	0.97
I-150×150×5-B2	0.98	0.98	1.02
I-150×120×6-B1	0.99	0.97	1.03
I-150×120×6-B2	0.92	0.91	1.00
I-150×100×6-B1	1.01	0.99	1.05
I-150×100×6-B2	0.97	0.93	1.01
Mean	0.97	0.95	1.01
COV	0.03	0.03	0.03

Figure 3.20: Test and FE load–end shortening curves for stub column specimen I-150×150×5-A2

Figure 3.21: Normalised test and FE moment–curvature curves for beam specimen I-150×150×5-B2

3.3.4 Parametric studies

Based on the validated FE models, parametric studies were performed to generate further numerical data over a wide range of cross-section aspect ratios and dimensions. The average material properties measured from the previous tensile coupon tests on 5 mm thick plates and the initial local imperfection magnitude of $t/10$ were employed throughout the present numerical parametric studies. Regarding the cross-section geometric dimensions of the modelled I-section stub columns and beams, the outer section heights were fixed at 150 mm and the flange widths were varied from 75 mm to 150 mm, with the plate thicknesses set to be equal and ranged between 2 mm and 17 mm, leading to a range of cross-section aspect ratios and dimensions being considered. The length of each stub column FE model was equal to three times the outer section height (i.e. 450 mm), while the flexural span lengths of the beam FE models were fixed at 1800 mm, with the two loading points located at third-points of the beam flexural span. In total, 245 numerical data on high-chromium stainless steel welded I-section stub columns and beams were generated through parametric studies.

3.4 Evaluation of existing design methods

3.4.1 General

In this section, the obtained test and numerical results were analysed, and then used to evaluate the applicability of the current Eurocode EN 1993-1-4 (CEN, 2015) and American design guide AISC 360 (AISC, 2013) and the continuous strength method (Afshan & Gardner, 2013; Zhao et al., 2017) to the new high-chromium stainless steel welded I-section stub columns and beams. Note that all the partial safety factors adopted in the existing design codes and the CSM were taken as unity herein, resulting in unfactored design resistances being evaluated. Table 3.9 reports the mean ratio of test and numerical to predicted cross-section resistances and the corresponding coefficient of variation (COV) for each of the three considered design methods.

3.4.2 Eurocode EN 1993-1-4 (EC3)

The current Eurocode EN 1993-1-4 (CEN, 2015) employs the concept of cross-section classification and an elastic, perfectly plastic material model for the design of stainless steel welded I-sections failing by local buckling. The class of a welded I-section is defined based on its most slender constituent plate element without considering the favourable element interaction effect, while classification of a plate element is determined through comparison of the width-to-thickness ratio $c/(t\varepsilon)$ against the EC3 codified slenderness limits, where c and t are respectively the flat width and thickness of the considered plate element (see Figure 3.10) and $\varepsilon = [(235/\sigma_{0.2})(E/210000)]^{0.5}$. Note that the current EN 1993-1-4 (CEN, 2015) employs two sets of slenderness limits respectively for outstand and internal plate elements, and within each set, the slenderness limits are also dependent on the applied loading and thus different for the same plate element subjected to varying loading. In the following sub-sections, the applicability of the EC3 slenderness limits to the new high-chromium stainless steel welded I-sections was assessed.

3.4.2.1 Class 3 slenderness limits

Class 3 slenderness limits distinguish slender sections, where local buckling occurs before the material yield stress is reached, from their non-slender counterparts, where the failure stresses exceed the yield stress and thus the ultimate cross-section compression and bending moment resistances are respectively greater than the yield loads ($A\sigma_{0.2}$) and elastic moment capacities (M_{el}). The beam and stub column test and numerical results are normalised by the cross-section elastic moment capacities and yield loads, respectively, and then plotted against the $c/(t\varepsilon)$ ratios of the webs of the considered I-sections in Figures 3.22 and 3.23, together with the EC3 Class 3 slenderness limits for internal elements subjected to bending and compression ($c/(t\varepsilon)=90$ and $c/(t\varepsilon)=37$, respectively). The comparison results indicated that the existing EC3 Class 3 slenderness limits for internal elements are applicable to the new high-chromium stainless steel welded I-sections. A similar graphical comparison was also performed for the outstand flanges under compression, based on the stub

column and beam test and numerical data, as shown in Figure 3.24, also revealing the applicability of the existing EC3 Class 3 slenderness limit for outstand elements under compression ($c/(t\epsilon)=14$). For welded I-sections bent about the minor principal axis, the outstand flanges are subjected to compressive stress gradients, with the corresponding EC3 Class 3 slenderness limit of $21\sqrt{k_\sigma}$, where k_σ is the buckling factor and equal to 0.57 for a triangular stress distribution (CEN, 2006). The EC3 Class 3 slenderness limit for outstand elements subjected to compressive stress gradients was also shown to capture the test and numerical data, as evident in Figure 3.25. In sum, the existing Class 3 slenderness limits set out in EN 1993-1-4 (CEN, 2015) are applicable to the new high-chromium stainless steel welded I-sections.

Figure 3.22: EN 1993-1-4 Class 3 slenderness limit for internal elements in bending

Figure 3.23: EN 1993-1-4 Class 3 slenderness limit for internal elements in compression

Figure 3.24: EN 1993-1-4 Class 3 slenderness limit for outstand elements in compression

Figure 3.25: EN 1993-1-4 Class 3 slenderness limit for outstand elements subjected to triangular compressive stress distribution

3.4.2.2 Class 2 slenderness limits

Cross-sections defined as Class 2 are capable of developing plastic moment capacities M_{pl} , in comparison with Class 3 cross-sections, where only elastic moment capacities M_{el} can be achieved. The test and numerical failure moments were thus normalised by the corresponding plastic moment capacities and plotted against the slenderness parameters $c/(t\epsilon)$ of the internal webs and outstand flanges of the I-sections, respectively, as shown in Figures 3.26 and 3.27, allowing the applicability of the EC3 Class 2 slenderness limits for internal elements subjected to bending ($c/(t\epsilon)=76$) and outstand elements subjected to compression ($c/(t\epsilon)=10$) to be assessed. The comparison results again indicated that the current EC3 Class 2 slenderness limits are applicable to high-chromium stainless steel welded I-sections. The current Eurocode EN 1993-1-4 (CEN, 2015) contains no specific requirements on deformation capacities for Class 1 sections, and also does not allow for plastic design; therefore, the applicability of the EC3 Class 1 slenderness limits to the new high-chromium stainless steel welded I-sections is not discussed herein.

Figure 3.26: EN 1993-1-4 Class 2 slenderness limit for internal elements in bending

Figure 3.27: EN 1993-1-4 Class 2 slenderness limit for outstand elements in compression

3.4.2.3 Evaluation of EC3 design cross-section resistances

In this section, the accuracy of the EC3 design cross-section resistances was evaluated against the obtained test and numerical results. Regarding the cross-section bending resistances, the current EN 1993-1-4 (CEN, 2015) prescribes the use of the plastic (M_{pl}) and elastic (M_{el}) moment resistances for Class 1 (or 2) and Class 3 non-slender welded I-sections, respectively, while the cross-section compression resistances for Class 1, 2 and 3 non-slender welded I-sections are taken as the yield load ($A\sigma_{0.2}$). For Class 4 slender cross-sections, the effective width method is adopted in EN 1993-1-4 (CEN, 2015) to consider local buckling and its effect on the cross-section resistance. The effective width method treats the plated section as an assemblage of isolated plates without accounting for the beneficial effect of element interaction, and considers loss of effectiveness due to local buckling through a reduction in plate element width. The effective width of each plate element is calculated as the product of the original plate element flat width c and the reduction factor ρ , given by Equation (3.5), where $\bar{\lambda}_l$ is the plate element slenderness, as defined by Equation (3.6), where k_σ is the buckling factor, taken as 0.43 and 4.0 for outstand and internal plate elements in uniform compression (CEN, 2006), respectively.

$$\rho = \begin{cases} \frac{0.772}{\bar{\lambda}_l} - \frac{0.079}{\bar{\lambda}_l^2} & \text{but } \leq 1.0 \quad \text{for internal elements} \\ \frac{1}{\bar{\lambda}_l} - \frac{0.188}{\bar{\lambda}_l^2} & \text{but } \leq 1.0 \quad \text{for outstand elements} \end{cases} \quad (3.5)$$

$$\bar{\lambda}_l = \frac{c/t}{28.4\sqrt{k_\sigma(235/\sigma_{0.2})}} \quad (3.6)$$

Upon determination of the effective widths of all the constituent plate elements (web and flanges) of the welded I-section, the effective cross-section area A_{eff} and modulus W_{eff} can be determined. Note that iteration is generally required in the calculation of cross-section effective modulus W_{eff} , owing to the shift in the effective neutral axis.

The ratios of the test and numerical to EC3 predicted failure loads and moments are plotted against the slendernesses of the most slender constituent plate elements of the welded I-sections, and shown in Figure 3.28. Quantitative evaluation of the EC3 design cross-section bending and compression resistances is presented in Table 3.9. Both the graphical and quantitative evaluation results generally indicated that the current EN 1993-1-4 (CEN, 2015) yields conservative and scattered cross-section resistance predictions for the new high-chromium stainless steel welded I-sections, owing mainly to the neglect of material strain hardening of stainless steel and element interaction within I-section.

Table 3.9: Comparisons of test and numerical results with EC3, ASIC and CSM resistance predictions

(a) Stub column			
No. of test data: 8	$N_u/N_{u,EC3}$	$N_u/N_{u,AISC}$	$N_u/N_{u,CSM}$
No. of FE data: 133			
Mean	1.17	1.20	1.05
COV	0.12	0.13	0.02
(b) Beam in major-axis bending			
No. of test data: 4	$M_u/M_{u,EC3}$	$M_u/M_{u,AISC}$	$M_u/M_{u,CSM}$
No. of FE data: 66			
Mean	1.20	1.20	1.10
COV	0.11	0.13	0.06
(c) Beam in minor-axis bending			
No. of test data: 4	$M_u/M_{u,EC3}$	$M_u/M_{u,AISC}$	$M_u/M_{u,CSM}$
No. of FE data: 46			
Mean	1.58	1.33	1.07
COV	0.10	0.08	0.08

Figure 3.28: Comparison of test and numerical results with EC3 resistance predictions

3.4.3 American design guide AISC 27 (AISC)

The American design guide AISC 27 (AISC, 2013) also employs the concept of cross-section classification and an elastic, perfectly plastic material model for the treatment of local buckling. AISC 27 (AISC, 2013) adopts the same slenderness limits for cross-section classification as those set out in EN 1993-1-4 (CEN, 2015). Cross-sections under compression are categorised as non-slender and slender sections in AISC 27 (AISC, 2013), which correspond to Class 1, 2 and 3 and Class 4 sections defined in EN 1993-1-4 (CEN, 2015), respectively, while cross-sections in bending are classified as compact, non-compact and slender sections in AISC 27 (AISC, 2013), corresponding to Class 1 and 2, Class 3 and Class 4 sections specified in EN 1993-1-4 (CEN, 2015), respectively. The AISC design resistance of a welded I-section under compression is calculated as $N_{u,AISC} = Q_a Q_s A \sigma_{0.2}$, where Q_s is the net section reduction factor for slender unstiffened plate elements (i.e. outstand flange), as given by

Equation (3.7), in which b_p is equal to half of the flange width for I-shaped sections, and Q_a is the net section reduction factor for slender stiffened plate elements (i.e. internal web), as given by Equation (3.8), in which A_e is the sum of the effective area of each constituent plate element. Note that cumbersome iterations are generally required in the determination of A_e (and thus Q_a , F_{cr} and $N_{u,AISC}$).

$$Q_s = \begin{cases} 1.0 & \text{for } \frac{b_p}{t} \leq 0.47 \sqrt{\frac{E}{\sigma_{0.2}}} \\ 1.498 - 1.06 \frac{b_p}{t} \sqrt{\frac{\sigma_{0.2}}{E}} & \text{for } 0.47 \sqrt{\frac{E}{\sigma_{0.2}}} < \frac{b_p}{t} \leq 0.90 \sqrt{\frac{E}{\sigma_{0.2}}} \\ \frac{0.44E}{\sigma_{0.2} (b_p/t)^2} & \text{for } \frac{b_p}{t} > 0.90 \sqrt{\frac{E}{\sigma_{0.2}}} \end{cases} \quad (3.7)$$

$$Q_a = \frac{A_e}{A} \quad (3.8)$$

Regarding the cross-section bending moment resistances, AISC 27 (AISC, 2013) adopts the plastic moment capacities for compact I-sections and considers partial plasticity in the determination of bending moment resistances for non-compact I-sections, while an elastic critical buckling moment approach is adopted in AISC 27 (AISC, 2013) for the calculation of cross-section bending moment resistances of slender I-sections, with the design formulae given in Clauses 6.2 of AISC 27 (AISC, 2013). The accuracy of the current American design guide AISC 27 (AISC, 2013) for high-chromium stainless steel welded I-section stub columns and beams was assessed through comparisons of the obtained test and numerical load-carrying capacities against the AISC resistance predictions, as shown in Table 3.9. The results of the comparisons generally indicated that AISC 27 (AISC, 2013) yields more accurate design cross-section bending moment resistances than EN 1993-1-4 (CEN, 2015), but with more conservative and scattered predicted compression resistances.

3.4.4 Continuous strength method (CSM)

The continuous strength method (Afshan & Gardner, 2013; Zhao et al., 2017; Zhao & Gardner, 2018) is a recently proposed deformation-based design method that offers an alternative way to the traditional cross-section classification framework employed in the current design standards and also enables a rational exploitation of material strain hardening through the use of an advanced bi-linear (elastic, linear hardening) material model. The first step of the application of the CSM lies in the calculation of the cross-section deformation capacity $\varepsilon_{csm}/\varepsilon_y$ from the CSM base curve (Afshan & Gardner, 2013; Zhao et al., 2017), as given by Equation (3.9), where ε_{csm} is the limiting (maximum attainable) strain of the cross-section subjected to the applied loading, $\varepsilon_y = \sigma_{0.2}/E$ is the material yield strain and $\bar{\lambda}_p = \sqrt{\sigma_{0.2}/\sigma_{cr}}$ is the cross-section slenderness, where σ_{cr} is the elastic critical local buckling stress of the full cross-section, and determined herein using the finite strip software CUFSM (Schafer & Ádány, 2006), with the beneficial element interaction effect within the cross-section considered. Note that $\bar{\lambda}_p = 0.68$ defines the boundary condition between non-slender and slender sections, at which point the limiting strain is equal to the material yield strain with $\varepsilon_{csm}/\varepsilon_y = 1$.

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \begin{cases} \frac{0.25}{\bar{\lambda}_p^{3.6}} \text{ but } \leq \min\left(15, \frac{C_1 \varepsilon_u}{\varepsilon_y}\right) & \text{for } \bar{\lambda}_p \leq 0.68 \\ \left(1 - \frac{0.222}{\bar{\lambda}_p^{1.05}}\right) \frac{1}{\bar{\lambda}_p^{1.05}} & \text{for } \bar{\lambda}_p > 0.68 \end{cases} \quad (3.9)$$

The CSM design stress σ_{csm} is dependent on the limiting strain of the cross-section and calculated using the CSM elastic, linear hardening material model, as shown in Figure 3.29, where C_1 , C_2 , C_3 and C_4 are the CSM material coefficients (Buchanan et al., 2016). Specifically, the material coefficient C_1 is used in Equation (3.9) to prevent over-predicting the design failure stresses, while the material coefficient C_2 is adopted in Equation (3.10) to define the strain hardening slope E_{sh} , which allows design failure stresses greater than the material yield stress to be obtained for non-slender cross-sections with $\varepsilon_{csm}/\varepsilon_y > 1$. The material coefficients C_3 and C_4 are used to predict

the failure strain $\varepsilon_u = C_3 (1 - \sigma_{0.2}/\sigma_u) + C_4$. With regard to austenitic stainless steel, C_1 , C_2 , C_3 and C_4 are taken as 0.1, 0.16, 1.0 and 0.0, respectively (Buchanan et al., 2016).

$$E_{sh} = \frac{\sigma_u - \sigma_{0.2}}{C_2 \varepsilon_u - \varepsilon_y} \quad (3.10)$$

Figure 3.29: CSM elastic, linear hardening material model

Upon calculation of the cross-section limiting strain ε_{csm} and the strain hardening slope E_{sh} , the CSM design stress σ_{csm} can then be derived from Equation (3.11), while the CSM design cross-section resistances under compression and bending ($N_{u,csm}$ and $M_{u,csm}$) are determined from Equations (3.12) and (3.13), respectively, where α is the CSM bending parameter, and equal to 2.0 and 1.2 for I-sections bent about the major and minor principal axes, respectively.

$$\sigma_{csm} = \begin{cases} \sigma_{0.2} + E_{sh} (\varepsilon_{csm} - \varepsilon_y) & \text{for } \varepsilon_{csm} \geq \varepsilon_y \\ E \varepsilon_{csm} & \text{for } \varepsilon_{csm} < \varepsilon_y \end{cases} \quad (3.11)$$

$$N_{u,csm} = A \sigma_{csm} \quad (3.12)$$

$$M_{u,csm} = \begin{cases} W_{pl} \sigma_{0.2} \left[1 + \frac{E_{sh}}{E} \frac{W_{el}}{W_{pl}} \left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1 \right) - \left(1 - \frac{W_{el}}{W_{pl}} \right) / \left(\frac{\varepsilon_{csm}}{\varepsilon_y} \right)^\alpha \right] & \text{for } \bar{\lambda}_p \leq 0.68 \\ W_{el} \sigma_{csm} & \text{for } \bar{\lambda}_p > 0.68 \end{cases} \quad (3.13)$$

Table 3.9 reports the mean ratios of the test and numerical to CSM design cross-section compression and bending resistances, indicating a high level of design accuracy and consistency of the CSM. Comparisons between the CSM and the two considered design standards (CEN, 2015; AISC, 2013) are shown in Figures 3.30 and 3.31, where the test and numerical compression and bending moment resistances normalised by the corresponding predicted cross-section resistances are plotted against the cross-section slendernesses, revealing that the CSM yields more accurate and less scattered cross-section resistances than the two design standards, which can be mainly attributed to the rational exploitation of material strain hardening of stainless steel and consideration of element interaction within I-section in the CSM.

Figure 3.30: Comparison of test and numerical results with CSM and EC3 resistance predictions

Figure 3.31: Comparison of test and numerical results with CSM and AISC resistance predictions

3.5 Reliability analysis

The reliability of the three considered design methods (EN 1993-1-4, AISC 27 and the CSM) when applied to the new high-chromium stainless steel welded I-section stub columns and beams was assessed herein, according to the requirements and procedures given in EN 1990 (CEN, 2002) and AISC 27 (AISC, 2013). In the present reliability analysis, the material over-strength ratio for austenitic stainless steel and the corresponding COV were taken as 1.3 and 0.06, respectively, and the COV of the geometric properties of stainless steel cross-sections was taken as 0.05, following the recommendations of Afshan et al. (2015). The key statistical parameters calculated according to EN 1990 (CEN, 2002) are summarised in Table 3.10, where $k_{d,n}$ is the design (ultimate limit state) fractile factor, b is the mean ratio of the test and numerical 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. Table 3.11 reports the key reliability analysis results for AISC 27 (AISC, 2013), including the professional factor P_m , the COV of the load effects V_Q , the COV of the resistances V_R , the reliability index β and the resistance factor ϕ_{AISC} . The resulting (required) partial safety factors for the CSM and codified EC3 design rules, as reported in Table 3.10, are both less than the current used value of 1.1 in EN 1993-1-4 (CEN, 2015), while the resulting (required) resistance factor for the AISC design rules, as presented in Table 3.11, is larger than the target value of 0.9 in AISC 27 (AISC, 2013), therefore demonstrating the reliability of the three considered design methods when applied to the new high-chromium stainless steel welded I-section stub columns and beams.

Table 3.10: 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_{M0}
EN 1993-1-4	261	3.128	1.379	0.187	0.203	1.07
CSM	261	3.128	1.050	0.071	0.106	1.02

Table 3.11: Reliability analysis results calculated according to AISC 27

Design method	No. of test and FE data	P_m	V_Q	V_R	β	ϕ_{AISC}
AISC 27	261	1.23	0.19	0.17	2.60	1.22

3.6 Concluding remarks

A thorough testing and numerical modelling programme has been conducted to investigate the material responses and local stability of welded I-sections made of the new grade EN 1.4420 high-chromium austenitic stainless steel. The testing programme included material testing, membrane residual stress measurements, initial local geometric imperfection measurements, eight stub column tests, four major-axis four-point bending tests and four minor-axis four-point bending tests. A new membrane residual stress predictive model for the new high-chromium stainless steel welded I-sections was proposed. The testing programme was supplemented by a parallel numerical modelling programme, where finite element models were firstly developed and validated against the obtained test data and then employed to perform parametric studies to generate further numerical data over a wide range of cross-section geometric dimensions. The obtained test and numerical results were adopted to assess the applicability of the cross-section slenderness limits and local buckling treatment, as given in both the Eurocode EN 1993-1-4 (CEN, 2015) and American design guide AISC 27 (AISC, 2013) to welded I-sections made of the new high-chromium stainless steel. The codified slenderness limits were found to be applicable to the new high-chromium stainless steel welded I-sections, while the codified treatment of local buckling was shown to yield conservative and scattered cross-section resistance predictions, owing principally to the lack of consideration of the favourable material strain hardening of stainless steel and element interaction within I-section. The accuracy of the continuous strength method (Afshan & Gardner, 2013; Zhao et al., 2017) for the design of the new high-chromium stainless steel welded I-sections was also assessed. The CSM accounts for the beneficial effects of material strain hardening and element interaction in calculating cross-section compression and bending resistances, and was shown to yield substantially improved design accuracy and consistency over the existing design standards (CEN, 2015; AISC, 2013). Finally, statistical analyses were performed to confirm the reliability of the design rules given in EN 1993-1-4 (CEN, 2015) and AISC 27 (AISC, 2013) as well as the CSM (Afshan & Gardner, 2013; Zhao et al., 2017).

CHAPTER 4

CROSS-SECTION BEHAVIOUR OF S690 HIGH STRENGTH STEEL WELDED I-SECTIONS

4.1 Introduction

This chapter presents a comprehensive experimental and numerical study of the material properties and cross-section load-carrying capacities of S690 high strength steel welded I-section stub columns and beams. An experimental programme was firstly conducted on eight S690 high strength steel I-sections welded by GMAW, including material testing, membrane residual stress measurements, initial local geometric imperfection measurements, sixteen stub column tests and twelve in-plane four-point bending tests, with six performed about the cross-section major principal axis and six conducted about the cross-section minor principal axis. The experimental programme was supplemented by a numerical modelling programme, where finite element models were firstly developed to simulate the experimental observations and then adopted to perform parametric studies to generate further numerical data over a wide range of cross-section aspect ratios and dimensions. Finally, the experimentally and numerically obtained results were employed to assess the accuracy of the existing slenderness limits and local buckling design rules for S690 high strength steel welded I-section stub columns and beams, as set out in the European code EN 1993-1-12 (CEN, 2007), American specification ANSI/AISC 360-16 (AISC, 2016) and Australian standard AS 4100 (AS, 2016). The findings of this research work have been reported in Sun et al. (2019a, 2019b).

4.2 Experimental study

4.2.1 General

A testing programme was carried out to investigate the material properties, membrane residual stresses and compression and bending moment resistances of S690 high strength steel welded I-sections. Eight I-section sizes – I-50×50×5, I-70×70×5, I-80×60×5, I-90×70×5, I-100×100×5, I-140×70×5, I-150×150×5, I-200×100×5 – were considered in this testing programme, and all the I-sections were fabricated from the same batch of 5 mm thick S700MC high strength steel hot-rolled plates by means of robotic GMAW. The detailed welding process is described as follows. Three steel plates were positioned on the flat work bench to form a well-aligned I-shaped profile. Two robotic arms then started welding at the same time from different sides of the web-to-flange junctions at different ends, as shown in Figure 4.1. This welding strategy helped minimise the member distortion during welding. It is worth noting that the non-copper coated, low alloy (0.3 Cr, 1.4% Ni and 0.25% Mo) electrode OK AristoRod 69, with the ultimate stress equal to 800 MPa, was used for welding. Overall, the testing programme included material tensile coupon tests to obtain the stress–strain responses of the studied S690 high strength steel, membrane residual stress measurements to determine the membrane residual stress distributions and amplitudes in S690 high strength steel welded I-sections, imperfection measurements to derive the initial local geometric imperfections of the stub column and beam specimens, concentric compression tests to investigate the cross-section compression resistances and local stability of S690 high strength steel welded I-section stub columns, and in-plane four-point bending tests to study the in-plane flexural behaviour and resistances of S690 high strength steel welded I-section beams.

Figure 4.1: Robotic GMAW setup

4.2.2 Material testing and membrane residual stress measurements

Material testing was firstly conducted to derive the material stress–strain responses of the studied S690 high strength steel. Tensile coupons were extracted from the same batch of plates as that used in the fabrication of the welded I-section stub column and beam specimens. Specifically, two longitudinal coupons were cut along the rolling direction of the plate, while two additional transverse coupons were extracted perpendicularly to the plate rolling direction. The dimensions of the two pairs of coupons are in compliance with the geometric requirements specified in EN ISO 6892-1 (CEN, 2016b). The test setup and procedures were the same as those employed for the material tensile coupon tests on the studied high-chromium austenitic stainless steel, as detailed in Section 3.2.2. Figure 4.2 shows the material stress–strain curves measured from both the longitudinal and transverse coupons, while the key average measured material properties, including the Young’s modulus E , the yield stress f_y , the ultimate stress f_u , the ultimate-to-yield stress ratio f_u/f_y , the strain at the ultimate stress ε_u , and the fracture strain measured over the standard gauge length of 50 mm ε_f , are reported in Table 4.1. It is worth noting that the material stress–strain curves of grade S690 high strength steel display much shorter yield plateaux and lower levels of material strain hardening and ductility (as also reflected by the ratios of f_u/f_y and ε_u in Table 4.1), compared to those exhibited by its normal strength mild steel counterparts.

Figure 4.2: Measured stress–strain curves for studied S690 high strength steel

Table 4.1: Average tensile material properties measured from longitudinal and transverse coupons

Direction	E (MPa)	f_y (MPa)	f_u (MPa)	ϵ_u (%)	ϵ_f (%)	f_u/f_y
Longitudinal	216290	702.6	750.3	11	24	1.07
Transverse	201810	701.8	765.6	10	24	1.09

Following the material testing, the membrane residual stresses in the studied S690 high strength steel welded I-sections were measured and reported herein. The sectioning method was employed for the membrane residual stress measurements on two S690 high strength steel welded I-sections I-200×100×5 and I-150×150×5, with the procedures being in line with those described in Section 3.2.3. The locations of the strips sectioned for the membrane residual stress measurements are depicted in Figure 4.3; for each strip, the nominal length and width are equal to 150 mm and 10 mm, respectively. Figure 4.4 depicts a typical sectioned S690 high strength steel welded I-section I-200×100×5.

Figure 4.3: Locations and dimensions of strips within I-sections (dimensions in mm)

Figure 4.4: Typical sectioned welded I-section I-200×100×5

Based on the measured data obtained from the sectioning process, the membrane residual stresses can be determined, following the procedures reported in Section 3.2.3. The distributions and amplitudes of the experimentally obtained membrane residual stresses for the two S690 high strength steel welded I-sections I-200×100×5 and I-150×150×5 are shown in Figures 4.5(a) and 4.5(b), respectively, while the peak values of both the obtained tensile and compressive residual stresses, normalised by

the material yield stress f_y measured from the longitudinal coupons, are reported in Table 4.2, where the subscripts ‘t’ and ‘c’ respectively indicate tensile and compressive residual stresses, and the subscripts ‘f’ and ‘w’ denote flange and web, respectively. Note that positive and negative values indicate tensile and compressive membrane residual stresses, respectively. Given that there have been no codified membrane residual stress predictive models for high strength steel welded I-sections, the corresponding predictive models for normal strength mild steel (S235, S275 and S355) welded I-sections, as set out in the European convention ECSS (1976) and Swedish regulations BSK 99 (2003), were assessed herein for their applicability to S690 high strength steel welded I-sections. The two codified residual stress predictive models for normal strength mild steel welded I-sections were established based on the same distribution pattern, as shown in Figure 3.8, but with different distribution coefficients (a , b , c and d) and peak compressive residual stresses (f_{fc} and f_{wc}), as presented in Table 4.3. A graphical comparison between the measured membrane residual stresses and the two predictive models given in ECSS (1976) and BSK 99 (2003) is depicted in Figures 4.6(a) and 4.6(b); note that the measured residual stress data points are plotted in a normalised format, with the normalised positions at 0.0 (origin point) and 1.0 (end point) respectively representing the web-to-flange junction and the mid-point of the web or the tip of the flange. The comparison results indicated that the two codified predictive models generally result in unduly conservative peak tensile residual stress predictions but over-predicted peak compressive residual stresses for S690 high strength steel welded I-sections. It is also evident in Figure 4.6 that the transition regions of the membrane residual stresses in S690 high strength steel welded I-sections were considerably wider when compared to those specified by the two codified predictive models for normal strength mild steel welded I-sections. This thus prompted the development of a new predictive model specifically for predicting the membrane residual stresses in S690 high strength steel welded I-sections. The new membrane residual stress predictive model follows the general

pattern shown in Figure 4.5, but employs a new set of distribution coefficients and different peak residual stress magnitudes. Figure 4.6 shows a comparison of the new residual stress predictive model with the measured membrane residual stresses in S690 high strength steel welded I-sections, indicating good agreement.

Figure 4.5: Measured membrane residual stress patterns and amplitudes

Table 4.2: Measured peak membrane residual stresses of S690 high strength steel welded I-sections

Specimen	Peak tensile residual stresses		Peak compressive residual stresses	
	Flange (f_{ft}/f_y)	Web (f_{wt}/f_y)	Flange (f_{fc}/f_y)	Web (f_{wc}/f_y)
I-200×100×5	0.48	0.38	-0.32	-0.28
	0.51		-0.36	
I-150×150×5	0.43	0.26	-0.30	-0.29
	0.53		-0.35	
Mean	0.49	0.32	-0.33	-0.29
Maximum	0.53	0.38	-0.36	-0.29

(a) flange

(b) web

Figure 4.6: Comparisons of measured residual stress patterns and amplitudes with predictive models

Table 4.3: Membrane residual stress predictive models for S690 high strength steel welded I-sections

Residual stress predictive model	$f_{ft} = f_{wt}$	$f_{fc} = f_{wc}$	a	b	c	d
ECCS (1976)	$1.0f_y$	$0.25f_y$	$0.05b_f$	$0.15b_f$	$0.075h_w$	$0.05h_w$
BSK 99 (2003)	$1.0f_y$	From equilibrium	$0.75t_f$	$1.5t_f$	$1.5t_w$	$1.5t_w$
Proposed model	$0.8f_y$	From equilibrium	$0.225b_f$	$0.15b_f$	$0.075h_w$	$0.225h_w$

4.2.3 Initial local geometric imperfection measurements

Initial local geometric imperfections may greatly affect the local stability and load-carrying capacities of thin-walled section steel structural components, and were thus measured for all the S690 high strength steel welded I-section specimens herein. The test setup and procedures were the same as those described in Section 3.2.4. The initial geometric imperfection magnitudes of the web and two flanges (ω_w , ω_{f1} and ω_{f2}) were taken as the largest values from the three measurements on the respective plate elements, as reported in Tables 4.4 and 4.6, while the initial local geometric imperfection magnitude of the specimen ω_0 was taken as the maximum of ω_w , ω_{f1} and ω_{f2} .

4.2.4 Stub column tests

For each of the eight S690 high strength steel welded I-sections, two repeated stub column tests were carried out, in order to investigate their cross-section compression resistances and local stability. The nominal length of each stub column specimen was taken as the minimum of three times the outer section height and twenty times the radius of gyration about the minor principal axis (Ziemian, 2010); each selected nominal specimen length was deemed long enough to contain a representative distribution pattern of membrane residual stresses and initial local geometric imperfections, but still short enough to prevent the occurrence of global instability.

The measured member length and cross-section dimensions of each S690 high strength steel welded I-section stub column specimen are reported in Table 4.4. All the stub column specimens were compressed between the fixed end platens of an INSTRON 2000 kN testing machine at a constant loading rate of 0.3 mm/min. The stub column test setup and procedures followed those described in Section 3.2.5. The load–end shortening curves for all the sixteen tested S690 high strength steel welded I-section stub columns are shown in Figure 4.7, while the key test results, including the failure load N_u , the end shortening at failure δ_u , and the ultimate-to-yield load ratio $N_u/(Af_y)$ are reported in Table 4.5. All the tested S690 high strength steel welded I-section stub columns were shown to fail by local buckling, and Figure 4.8 displays typical failure modes of the specimens I-150×150×5-1, I-90×70×5-1 and I-200×100×5-1 with the cross-section aspect ratios h/b_f varying from 1.0 to 2.0.

Table 4.4: Measured geometric dimensions and initial local geometric imperfections of S690 high strength steel welded I-section stub column specimens

Specimen ID	L (mm)	h (mm)	b_f (mm)	t (mm)	ω_w (mm)	ω_{f1} (mm)	ω_{f2} (mm)	ω_0 (mm)
I-50×50×5-1	149.5	47.7	49.4	4.88	0.04	0.05	0.04	0.05
I-50×50×5-2	149.1	49.5	49.4	4.88	0.03	0.03	0.03	0.03
I-70×70×5-1	206.7	68.7	69.2	4.97	0.10	0.07	0.09	0.10
I-70×70×5-2	207.3	68.2	69.1	4.96	0.07	0.03	0.06	0.07
I-80×60×5-1	239.8	80.0	60.0	5.00	0.03	0.09	0.08	0.09
I-80×60×5-2	239.9	80.0	59.9	4.99	0.13	0.08	0.06	0.13
I-90×70×5-1	265.6	90.0	69.2	5.00	0.05	0.05	0.04	0.05
I-90×70×5-2	265.5	89.9	69.1	4.97	0.05	0.06	0.04	0.06
I-100×100×5-1	299.9	99.4	98.9	4.89	0.07	0.10	0.12	0.12
I-100×100×5-2	296.1	99.2	99.0	4.99	0.07	0.11	0.10	0.11
I-140×70×5-1	298.9	139.4	69.1	5.00	0.08	0.04	0.04	0.08
I-140×70×5-2	300.0	139.4	69.0	4.99	0.07	0.09	0.07	0.09
I-150×150×5-1	450.0	149.9	149.0	4.91	0.11	0.10	0.14	0.14
I-150×150×5-2	448.5	149.8	149.0	4.95	0.09	0.12	0.10	0.12
I-200×100×5-1	417.2	199.5	99.4	4.96	0.10	0.08	0.07	0.10
I-200×100×5-2	417.5	199.4	99.4	4.98	0.11	0.07	0.07	0.11

Table 4.5: Key test results for S690 high strength steel welded I-section stub column specimens

Specimen ID	N_u (kN)	δ_u (mm)	$N_u/(Af_y)$
I-50×50×5-1	623.1	5.12	1.23
I-50×50×5-2	642.9	5.14	1.26
I-70×70×5-1	805.3	2.04	1.12
I-70×70×5-2	841.7	2.70	1.17
I-80×60×5-1	740.6	2.83	1.06
I-80×60×5-2	769.5	1.53	1.10
I-90×70×5-1	872.3	2.25	1.09
I-90×70×5-2	884.5	3.21	1.11
I-100×100×5-1	1127.3	1.53	1.10
I-100×100×5-2	1105.3	1.61	1.06
I-140×70×5-1	985.6	1.50	1.05
I-140×70×5-2	1023.8	1.46	1.09
I-150×150×5-1	1254.2	2.29	0.83
I-150×150×5-2	1274.6	1.67	0.84
I-200×100×5-1	1214.1	1.60	0.90
I-200×100×5-2	1155.4	1.35	0.85

Figure 4.7: Load–end shortening curves of tested S690 high strength steel welded I-section stub column specimens

Figure 4.8: Failure modes of S690 high strength steel welded I-section stub column specimens (from left to right: I-150×150×5-1, I-90×70×5-1 and I-200×100×5-1)

4.2.5 In-plane four-point bending tests

In-plane four-point bending tests were conducted on twelve S690 high strength steel welded I-section beams, with six for each principal axis, to investigate their in-plane flexural behaviour and bending moment resistances. The member lengths and cross-section geometric dimensions of the beam specimens were carefully measured and are reported in Table 4.6. All the beam tests were conducted under displacement control at a constant loading rate of 2 mm/min, with the test setups and procedures being in line with those detailed in Section 4.2.6. All the tested S690 high strength steel welded I-section beam specimens exhibited visible in-plane deformation and failed by local buckling; Figures 4.9 and 4.10 depict the failure modes for typical beam specimens I-100×100×5-MA (in major-axis bending) and I-100×100×5-MI (in minor-axis bending), respectively. The key test results, obtained from the four-point bending tests, are presented in Table 4.7, including the failure moment M_u , the ratios of M_u/M_{pl} and M_u/M_{el} , and the beam rotation capacity R . Figures 4.11 and 4.12 depict the normalised moment–curvature curves for all the tested S690 high strength steel welded I-section beam specimens in major-axis and minor-axis bending, respectively.

Table 4.6: Measured geometric dimensions and initial local geometric imperfections of S690 high strength steel welded I-section beam specimens

Specimen ID	L (mm)	h (mm)	b_f (mm)	t (mm)	ω_w (mm)	ω_{f1} (mm)	ω_{f2} (mm)	ω_0 (mm)
I-50×50×5-MA	496	49.44	49.42	4.97	0.04	0.13	0.08	0.13
I-70×70×5-MA	697	67.15	69.29	4.93	0.06	0.08	0.08	0.08
I-80×60×5-MA	694	79.39	59.01	4.96	0.03	0.14	0.11	0.14
I-90×70×5-MA	698	90.35	69.20	4.93	0.06	0.11	0.10	0.11
I-100×100×5-MA	999	99.02	99.09	4.91	0.05	0.15	0.13	0.15
I-140×70×5-MA	1397	139.33	69.23	4.94	0.04	0.09	0.08	0.09
I-50×50×5-MI	498	49.41	49.59	4.99	0.04	0.09	0.10	0.10
I-70×70×5-MI	874	68.62	69.28	4.97	0.07	0.12	0.07	0.12
I-80×60×5-MI	696	79.63	59.16	4.90	0.06	0.13	0.13	0.13
I-90×70×5-MI	756	90.35	69.12	4.93	0.06	0.09	0.13	0.13
I-100×100×5-MI	997	99.31	99.48	4.98	0.05	0.15	0.13	0.15
I-140×70×5-MI	1398	139.59	68.98	4.94	0.09	0.07	0.10	0.10

Table 4.7: Key test results for S690 high strength steel welded I-section beam specimens

Specimen ID	Axis of bending	M_u (kNm)	M_u/M_{pl}	M_u/M_{el}	R
I-50×50×5-MA	Major axis	10.0	1.10	1.30	>1.61
I-70×70×5-MA		19.4	1.09	1.25	>4.04
I-80×60×5-MA		21.0	1.07	1.24	>2.41
I-90×70×5-MA		27.7	1.07	1.22	2.68
I-100×100×5-MA		39.8	1.02	1.15	1.48
I-140×70×5-MA		49.5	1.06	1.24	2.78
I-50×50×5-MI	Minor axis	5.0	1.11	1.73	>4.15
I-70×70×5-MI		9.3	1.08	1.66	>6.13
I-80×60×5-MI		6.8	1.08	1.69	>6.16
I-90×70×5-MI		9.2	1.07	1.67	>5.35
I-100×100×5-MI		17.2	0.98	1.49	—*
I-140×70×5-MI		10.0	1.10	1.80	>8.76

* The failure moment M_u of I-100×100×5-MI is lower than the cross-section plastic moment capacity M_{pl} .

Figure 4.9: Test and FE failure modes for S690 high strength steel beam specimen I-100×100×5-MA in major-axis bending

Figure 4.10: Test and FE failure modes for S690 high strength steel beam specimen I-100×100×5-MI in minor-axis bending

Figure 4.11: Normalised moment–curvature curves for S690 high strength steel beam specimens in major-axis bending

Figure 4.12: Normalised moment–curvature curves for S690 high strength steel beam specimens in minor-axis bending

4.3 Numerical study

4.3.1 General

In parallel with the testing programme, a numerical modelling programme, including a validation study and a parametric study, was conducted using the finite element analysis software ABAQUS (2014). In the numerical validation study, FE models were developed and validated against the test results, while in the parametric study, the validated FE models were employed to generate further numerical data over a wide range of cross-section aspect ratios and dimensions to supplement the obtained test results.

4.3.2 Development and validation of finite element models

The same modelling assumptions, procedures and techniques, as those described in Section 3.3.2, were employed herein to develop the FE models on S690 high strength steel welded I-section stub columns and beams. Figure 4.13 shows a typical membrane residual stress pattern incorporated into the FE models for the S690 high strength steel welded I-section stub column specimens I-150×150×5-1 and I-150×150×5-2. Regarding the incorporation of initial local geometric imperfections into the S690 high strength steel welded I-section stub column and beam FE models. Four initial local imperfection magnitudes, including ω_0 , $t/10$, $t/30$ and $t/100$, were employed to factor the imperfection patterns for the purpose of assessing the sensitivity of the FE models to imperfections and seeking the most appropriate local geometric imperfection magnitudes to be used in the parametric studies.

Figure 4.13: Typical membrane residual stress distribution (in N/mm²) in modelled I-150×150×5 (Positive values indicate tensile residual stresses while negative values indicate compressive residual stresses)

Upon development of the S690 high strength steel welded I-section stub column and beam FE models, nonlinear Riks analysis (ABAQUS, 2014) was conducted to determine the numerical failure loads, load–deformation curves and failure modes, which were then compared against the corresponding test results, allowing the accuracy of the developed FE models to be examined. Tables 4.8 and 4.9 report the numerical to test failure load ratios for the S690 high strength steel welded I-section stub column and beam specimens, respectively; the comparison results revealed that all the four considered initial local geometric imperfection magnitudes lead to accurate and consistent predictions of the test failure loads, while the best agreement between the test and numerical failure loads was generally obtained when the imperfection magnitude equal to $t/100$ was employed in the numerical modelling. Figure 4.14 depicts a comparison between the test and FE load–end shortening curves for a typical S690 high strength steel welded I-section stub column specimen I-140×70×5-1, while the test and FE normalised moment–curvature curves for two typical beam specimens I-140×70×5-MA (in major-axis bending) and I-100×100×5-MI (in minor-axis bending) are compared in Figures 4.15 and 4.16, respectively; the comparison results indicated that the test load–deformation curves can be well captured by their numerical counterparts. Moreover, the numerical load–deformation

curves obtained from the numerical models with no incorporation of membrane residual stresses are also plotted in Figures 4.14, 4.15 and 4.16, and shown to almost coincide with the corresponding curves derived from the numerical models with the membrane residual stresses incorporated; this indicated that membrane residual stresses have an insignificant effect on the local stability and cross-section load-carrying capacities of S690 high strength steel welded I-sections. Excellent agreement was also obtained between the test and numerical failure modes, as shown in Figures 4.17, 4.9 and 4.10. To conclude, the developed FE models are capable of accurately simulating the stub column and beam tests on S690 high strength steel welded I-sections, and thus considered to be validated.

Table 4.8: Comparison of FE failure loads with test failure loads for varying initial local geometric imperfection magnitudes

Specimen ID	Finite element N_u / Test N_u			
	ω_0	$t/10$	$t/30$	$t/100$
I-50×50×5-1	0.90	0.85	0.88	0.90
I-50×50×5-2	0.90	0.85	0.87	0.89
I-70×70×5-1	0.97	0.91	0.95	0.98
I-70×70×5-2	0.95	0.90	0.93	0.95
I-80×60×5-1	0.96	0.92	0.95	0.97
I-80×60×5-2	0.92	0.90	0.92	0.94
I-90×70×5-1	0.93	0.88	0.91	0.93
I-90×70×5-2	0.91	0.87	0.89	0.91
I-100×100×5-1	0.93	0.89	0.92	0.94
I-100×100×5-2	0.96	0.91	0.95	0.97
I-140×70×5-1	0.97	0.92	0.95	0.98
I-140×70×5-2	0.94	0.89	0.92	0.94
I-150×150×5-1	0.99	0.96	0.98	1.00
I-150×150×5-2	0.98	0.95	0.97	0.99
I-200×100×5-1	0.95	0.92	0.94	0.96
I-200×100×5-2	1.00	0.96	0.99	1.01
Mean	0.95	0.91	0.93	0.96
COV	0.03	0.04	0.03	0.03

Table 4.9: Comparison of FE failure moments with test failure moments for varying initial local geometric imperfection magnitudes

Specimen ID	Finite element M_u / Test M_u			
	ω_0	$t/10$	$t/30$	$t/100$
I-50×50×5-MA	0.98	0.97	0.98	0.98
I-70×70×5-MA	0.95	0.94	0.94	0.95
I-80×60×5-MA	0.97	0.95	0.97	0.98
I-90×70×5-MA	1.01	1.00	1.01	1.02
I-100×100×5-MA	0.97	0.95	0.97	0.98
I-140×70×5-MA	1.00	0.98	0.99	1.00
I-50×50×5-MI	1.02	0.97	1.01	1.03
I-70×70×5-MI	0.96	0.89	0.95	0.98
I-80×60×5-MI	0.93	0.90	0.93	0.94
I-90×70×5-MI	0.91	0.86	0.89	0.91
I-100×100×5-MI	0.96	0.92	0.95	0.98
I-140×70×5-MI	0.96	0.90	0.94	0.97
Mean	0.97	0.94	0.96	0.98
COV	0.03	0.05	0.03	0.03

Figure 4.14: Test and FE load–end shortening curves for S690 high strength steel stub column specimen I-140×70×5-1

Figure 4.15: Normalised test and FE moment–curvature curves for S690 high strength steel beam specimen I-140×70×5-MA

Figure 4.16: Normalised test and FE moment–curvature curves for S690 high strength steel beam specimen I-100×100×5-MI

Figure 4.17: Test and FE failure modes for S690 high strength steel stub column specimen I-150×150×5-1

4.3.3 Parametric studies

The validated stub column and beam FE models were adopted to perform numerical parametric studies in this section, in order to expand the test data pool over a broader range of cross-section aspect ratios and dimensions. In the present parametric studies, the average measured material properties of the longitudinal coupons, together with the initial local geometric imperfection magnitudes equal to $t/100$, were used. Regarding the geometric dimensions of the modelled I-sections, the outer cross-section heights were fixed at 150 mm, while the flange widths were taken as 75 mm, 90 mm, 100 mm and 150 mm, respectively, leading to a wide range of cross-section aspect ratios being considered. The thicknesses of the flanges and web of each modelled I-section were set to be equal and varied between 2 mm and 15 mm, resulting in a broad range of cross-section dimensions being examined. The lengths of the stub column FE models were equal to three times the outer cross-section heights, i.e. 450 mm, while the flexural spans of all the modelled I-section beams were equal to 1500 mm, with the two loading points positioned at the third-points of the flexural spans. In total, 376 numerical data on S690 high strength steel welded I-section stub columns and beams were generated through parametric studies.

4.4 Evaluation of existing design standards

4.4.1 General

In this section, the obtained test and numerical results were adopted to assess the accuracy of the codified design rules for S690 high strength steel welded I-section stub columns and beams. Three established design standards, including the European code EN 1993-1-12 (EC3) (CEN, 2007), American specification ANSI/AISC 360-16 (AISC) (AISC, 2016) and Australian standard AS 4100 (AS) (AS, 2016), were considered. The Eurocode EN 1993-1-12 (CEN, 2007) was developed specifically for high strength steels with grades greater than S460 up to S700, though mirroring most of the design rules set out in EN 1993-1-1 (CEN, 2014) for normal strength mild steels, while both ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016) provide design rules for both normal strength mild steels and high strength steels with grades up to S690. With regard to the design of stub columns and beams failing by local buckling, all the three considered design standards employ the cross-section classification framework and the effective width method. In the following Sections 4.4.2 and 4.4.3, the accuracy of the relevant design rules for S690 high strength steel welded I-section stub columns and beams, as given in the three considered design standards, was respectively evaluated based on the test and numerical results.

4.4.2 Design rules for S690 high strength steel welded I-section stub columns

4.4.2.1 Cross-section classification limits

All the three considered design standards, including the European code EN 1993-1-12 (CEN, 2007), American specification ANSI/AISC 360-16 (AISC, 2016) and Australian standard AS 4100 (AS, 2016), adopt the cross-section classification

framework for the design of S690 high strength steel welded I-sections under compression. The existing American specification ANSI/AISC 360-16 (AISC, 2016) and Australian standard AS 4100 (AS, 2016) categorise cross-sections in compression as non-slender and slender sections, where local buckling occurs upon and prior to the attainment of the material yield stress f_y , respectively, with the corresponding cross-section compression resistances taken as the yield loads Af_y and effective compression resistances $A_{eff}f_y$. Four classes of cross-sections are defined in the Eurocode EN 1993-1-12 (CEN, 2007): Class 1, 2 and 3 sections (corresponding to the non-slender sections specified in ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016)) are capable of attaining the yield loads Af_y at failure, while Class 4 sections (corresponding to the slender sections in ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016)) fail before the material yield stress f_y is achieved, limiting the design cross-section compression resistances to the effective compression resistances $A_{eff}f_y$. The class of a welded I-section in compression is defined based on its most slender constituent plate element, while each plate element (i.e. web or flange) is classified through comparing the flat width-to-thickness ratio (c_w/t or c_f/t) against the corresponding slenderness limits, in which c_w and c_f are the flat widths of the web and flange, respectively. Note that the flat element widths of flanges of I-shaped sections are defined as half of the full flange widths in ANSI/AISC 360-16 (AISC, 2016), while the flat element widths of webs are defined as the clear distances between the flanges. The slenderness limits between slender and non-slender plate elements are also known as Class 3 limits, limiting width-to-thickness ratios and yield slenderness limits in EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016), respectively. Table 4.10 reports the slenderness limits between slender and non-slender outstand flanges (and internal webs), as set out in the three design codes, where $\varepsilon_{EC3} = \sqrt{235/f_y}$, $\varepsilon_{AISC} = \sqrt{E/f_y}$ and $\varepsilon_{AS} = \sqrt{250/f_y}$ are the material parameters to reflect the difference in material strengths between high strength and normal strength steels, and $k_c = 4\sqrt{t/c_w}$ is a geometric coefficient to

consider the effect of web size on the slenderness limit of flange.

Table 4.10: Summary of EC3, AISC and AS slenderness limits between slender and non-slender plate elements in compression

Design standard	Outstand flanges	Internal webs
EN 1993-1-12 (CEN, 2007)	$14\varepsilon_{EC3}$	$42\varepsilon_{EC3}$
ANSI/AISC 360-16 (AISC, 2016)	$0.64k_c^{0.5}\varepsilon_{AISC}$	$1.49\varepsilon_{AISC}$
AS 4100 (AS, 2016)	$14\varepsilon_{AS}$	$35\varepsilon_{AS}$

The experimentally and numerically obtained failure loads of S690 high strength steel welded I-section stub columns are normalised by the corresponding cross-section yield loads Af_y , and then plotted against the $c_{fl}/(t\varepsilon_{EC3})$, $c_{fl}/(t\varepsilon_{AISC}k_c^{0.5})$ and $c_{fl}/(t\varepsilon_{AS})$ ratios of the outstand flanges of the studied welded I-sections in Figures 4.18–4.20, respectively, together with the EC3, AISC and AS slenderness limits for non-slender/slender outstand flanges in compression. The comparison results generally revealed that the slenderness limits for non-slender/slender outstand flanges in compression, as set out in all the three design standards, are safe and accurate when applied to S690 high strength steel welded I-section stub columns. Similar graphical comparisons are also presented in Figures 4.21–4.23 for the internal webs of the studied S690 high strength steel welded I-section stub columns, which also indicated the accuracy of the codified slenderness limits for non-slender/slender internal elements in compression. In summary, the slenderness limits, as given in the current EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016), are safe and accurate for the plate element and cross-section classifications of S690 high strength steel welded I-sections in compression.

Figure 4.18: EN 1993-1-12 Class 3 slenderness limit for outstand elements in compression

Figure 4.19: ANSI/AISC 360-16 slenderness limit for outstand elements in compression

Figure 4.20: AS 4100 slenderness limit for outstand elements in compression

Figure 4.21: EN 1993-1-12 Class 3 slender limit for internal elements in compression

Figure 4.22: ANSI/AISC 360-16 slenderness limit for internal elements in compression

Figure 4.23: AS 4100 slenderness limit for internal elements in compression

4.4.2.2 Cross-section compression resistances

The accuracy of the cross-section compression resistance predictions for S690 high strength steel welded I-section stub columns, as determined from EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016), was evaluated in this section. All the three design codes prescribe the use of the cross-section yield loads (Af_y) for non-slender (Class 1, 2 and 3) welded I-sections under compression, and employ the effective width methods for predicting the compressive load-carrying capacities of their slender (Class 4) counterparts. The effective width methods account for loss of effectiveness of each constituent plate element of a slender section susceptible to local buckling through a reduction in plate element widths. The formulae for determining the (reduced) effective widths of slender internal and outstand plate elements, as specified in EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016), are given by Equations (4.1)–(4.3), respectively, where $\bar{\lambda}_l$ is the plate element slenderness and can be calculated from Equation (4.4), in which c is the flat width of the plate element and taken as c_w and c_f for internal web and outstand flange of an I-section, and k_σ is the buckling factor, respectively taken as 4.0 and 0.43 for internal and outstand plate elements in compression (CEN, 2006), and $\bar{\lambda}_{r,int}$ and $\bar{\lambda}_{r,out}$ are the AISC limiting width-to-thickness ratios for internal and outstand plate elements – see Table 4.10. Upon calculation of the effective width of each slender plate element of the welded I-section, the effective cross-section area (A_{eff}) and the effective compression resistance ($A_{eff}f_y$) can then be derived.

$$c_{eff,EC3} = \begin{cases} c \left(\frac{1}{\bar{\lambda}_l} - \frac{0.22}{\bar{\lambda}_l^2} \right) \leq c & \text{for internal elements} \\ c \left(\frac{1}{\bar{\lambda}_l} - \frac{0.188}{\bar{\lambda}_l^2} \right) \leq c & \text{for outstand elements} \end{cases} \quad (4.1)$$

$$c_{eff,AISC} = \begin{cases} c \left(\frac{1.31\bar{\lambda}_{r,int}}{c/t} - \frac{0.31\bar{\lambda}_{r,int}^2}{(c/t)^2} \right) \leq c & \text{for internal elements} \\ c \left(\frac{1.49\bar{\lambda}_{r,out}}{c/t} - \frac{0.49\bar{\lambda}_{r,out}^2}{(c/t)^2} \right) \leq c & \text{for outstand elements} \end{cases} \quad (4.2)$$

$$c_{eff,AS} = \begin{cases} c \left(\frac{35}{c/(t\varepsilon_{AS})} \right) \leq c & \text{for internal elements} \\ c \left(\frac{14}{c/(t\varepsilon_{AS})} \right) \leq c & \text{for outstand elements} \end{cases} \quad (4.3)$$

$$\bar{\lambda}_l = \frac{c/t}{28.4\varepsilon_{EC3}\sqrt{k_\sigma}} \quad (4.4)$$

The test and numerical failure loads of S690 high strength steel welded I-section stub columns are normalised by the design cross-section compression resistances predicted from the three design standards (CEN, 2007; AISC, 2016; AS, 2016), and then plotted against the c_w/t ratios of the I-sections, as shown in Figures 4.24–4.26, while Table 4.11 reports the mean test and numerical to predicted cross-section compression resistance ratios $N_u/N_{u,pred}$ and the corresponding COVs. The results of both the graphical and quantitative evaluations generally indicated that all the three design standards yield rather accurate and consistent cross-section compression resistance predictions for both S690 high strength steel non-slender and slender welded I-section stub columns.

Table 4.11: Comparisons of test and numerical results with EC3, AISC and AS compression resistance predictions

Cross-section	No. of test data	No. of FE data	$N_u/N_{u,EC3}$		$N_u/N_{u,AISC}$		$N_u/N_{u,AS}$	
			Mean	COV	Mean	COV	Mean	COV
Non-slender (Class 1–3)	12	43	1.06	0.05	1.06	0.05	1.07	0.05
Slender (Class 4)	4	47	1.05	0.03	1.02	0.01	1.10	0.05
Total	16	90	1.06	0.04	1.04	0.04	1.08	0.05

Figure 4.24: Comparison of test and FE results with EC3 resistance predictions

Figure 4.25: Comparison of test and FE results with AISC resistance predictions

Figure 4.26: Comparison of test and FE results with AS resistance predictions

4.4.3 Design rules for S690 high strength steel welded I-section beams

4.4.3.1 Eurocode EN 1993-1-12

Regarding the design of S690 high strength steel welded I-sections in bending, EN 1993-1-12 (CEN, 2007) adopts the same concept of cross-section classification as that employed in EN 1993-1-1 (CEN, 2014). Four cross-section classes were specified in EN 1993-1-12 (CEN, 2007): Class 1 and 2 (plastic) sections in bending can achieve the plastic moment capacities (M_{pl}), Class 3 (elastic) sections in bending are capable of attaining the elastic moment capacities (M_{el}), and Class 4 (slender) sections fail before the material yield stresses f_y are attained, with the design bending moment resistances limited to the effective moment capacities (M_{eff}). The EC3 Class 3 and Class 2 slenderness limits for classifying internal and outstand plate elements

under various loading conditions (i.e. compression, bending and compressive stress gradients) are listed in Table 4.12.

Table 4.12: Established slenderness limits in EN 1993-1-12

Plate element type	Loading condition	Class 3 limit	Class 2 limit
Outstand flange	Compression	$14\varepsilon_{EC3}$	$10\varepsilon_{EC3}$
Internal web	Bending	$124\varepsilon_{EC3}$	$83\varepsilon_{EC3}$
Outstand flange	Compressive stress gradient	$21\varepsilon_{EC3}k_{\sigma}^{0.5}$	–

The Class 3 slenderness limits for outstand plate elements in compression and internal plate elements in bending were assessed, based on the test and numerical data on S690 high strength steel welded I-section beams bent about the major principal axis, with the graphical assessment results respectively depicted in Figures 4.27 and 4.28, where the test and numerical failure bending moments, normalised by the cross-section elastic moment resistances, are plotted against the $c_f/(t\varepsilon_{EC3})$ and $c_w/(t\varepsilon_{EC3})$ ratios of the considered S690 high strength steel welded I-sections, together with the EC3 Class 3 slenderness limits (normalised by ε_{EC3}) for outstand plate elements in compression $c_f/(t\varepsilon_{EC3})=14$ and internal plate elements in bending $c_w/(t\varepsilon_{EC3})=124$. It was generally found that the Class 3 slenderness limits established in EN 1993-1-12 (CEN, 2007) are safe but conservative when used for the classification of internal webs (in bending) and outstand flanges (in compression) of S690 high strength steel welded I-sections subjected to major-axis bending. The failure moments, obtained from the structural testing and numerical modelling on S690 high strength steel welded I-section beams in minor-axis bending, were then used to assess the accuracy of the current EC3 Class 3 slenderness limit for outstand plate elements under triangular compressive stress gradients ($c_f/(t\varepsilon_{EC3})=21k_{\sigma}^{0.5}$, where $k_{\sigma}=0.57$) in Figure 4.29. The assessment results generally revealed that the EC3 Class 3 slenderness limit for outstand plate elements under triangular compressive stress gradients is rather conservative though safe when used for the classification of outstand flanges of S690 high strength steel welded I-section beams in minor-axis bending.

Figure 4.27: EC3 Class 3 slenderness limit for outstand elements in compression

Figure 4.28: EC3 Class 3 slenderness limit for internal elements in bending

Figure 4.29: EC3 Class 3 slenderness limit for outstand elements under triangular compressive stress gradients

A similar graphical assessment was also performed on the established EC3 Class 2 slenderness limit for internal plate elements in bending ($c_w/(t\epsilon_{EC3})=83$), based on the test and numerical data on S690 high strength steel welded I-section beams under major-axis bending, as shown in Figure 4.30. The EC3 Class 2 slenderness limit for outstand plate elements in compression ($c_f/(t\epsilon_{EC3})=10$) was assessed, based on the obtained test and numerical data on S690 high strength steel welded I-section beams bent about both the major and minor principal axes, as depicted in Figure 4.31. The results of the graphical assessments in Figures 4.30 and 4.31 generally revealed that the Class 2 slenderness limits established in EN 1993-1-12 (CEN, 2007) are safe and accurate when used for the classifications of internal webs (in bending) and outstand flanges (in compression) of S690 high strength steel welded I-section beams.

Figure 4.30: EC3 Class 2 slenderness limit for internal elements in bending

Figure 4.31: EC3 Class 2 slenderness limit for outstand elements in compression

The EC3 predictions of cross-section bending moment capacities for S690 high strength steel welded I-section beams were assessed through comparisons against the obtained test and numerical failure bending moments. The current EN 1993-1-12 (CEN, 2007) prescribes the use of cross-section plastic (M_{pl}) and elastic (M_{el}) moment capacities as the design bending moment resistances for Class 1 (and 2) and Class 3 sections, respectively, and adopts the effective width method to predict the cross-section bending moment resistances for those slender Class 4 sections. The effective width method makes due allowance for loss of effectiveness owing to local buckling by reducing the flat widths of the slender constituent outstand and internal plate elements. The (reduced) effective widths of slender flanges (outstand elements) and webs (internal elements) of I-sections can be calculated from Equation (4.1) (CEN, 2006). Upon calculation of the effective widths of all the slender plate elements of the Class 4 welded I-section, the EC3 effective section modulus ($W_{eff,EC3}$) and effective bending moment capacity ($M_{eff,EC3}=W_{eff,EC3}f_y$) can then be derived; it is worth noting that cumbersome iterations may be involved in the calculation of $W_{eff,EC3}$ due to the shift in effective neutral axis along with each round of calculation.

The ratios of the test and numerical failure bending moments to the EC3 cross-section bending moment resistance predictions are plotted against the c/t ratios of the flanges of the examined I-sections, as depicted in Figure 4.32, while Table 4.13 presents the mean test and numerical to EC3 design cross-section bending resistance ratios $M_u/M_{u,EC3}$, together with the corresponding COVs, for different classes of S690 high strength steel welded I-sections in bending. The results of both the graphical and quantitative comparisons revealed that the cross-section bending moment resistances are well predicted by the current EN 1993-1-12 (CEN, 2007) for S690 high strength steel Class 1 (and Class 2) welded I-section beams bent about both the principal axes and Class 3 S690 high strength steel welded I-section beams in major-axis bending, while EN 1993-1-12 (CEN, 2007) yields overly conservative bending moment

resistance predications for S690 high strength steel Class 3 welded I-section beams bent about the minor principal axis and S690 high strength steel Class 4 welded I-section beams in both major-axis bending and minor-axis bending.

Figure 4.32: Comparison of test and numerical failure moments with EC3 bending moment resistance predictions

Table 4.13: Comparisons of test and numerical failure moments against EC3 predicted bending moment resistances

Axis of bending	Class of cross-section	No. of test data	No. of FE data	$M_u / M_{u,EC3}$	
				Mean	COV
Major axis	Class 1 and 2 I-section	5	34	1.07	0.02
	Class 3 I-section	0	59	1.15	0.03
	Class 4 I-section	1	92	1.28	0.06
	Sub-total	6	185	1.18	0.08
Minor axis	Class 1 and 2 I-section	5	25	1.09	0.02
	Class 3 I-section	1	26	1.57	0.03
	Class 4 I-section	0	56	1.52	0.04
	Sub-total	6	101	1.44	0.13
Total		12	286	1.26	0.18

4.4.3.2 Australian Standard AS 4100

The current Australian standard AS 4100 (AS, 2016), which provides design rules for both normal strength mild steels and high strength steels with grades up to S690, also employs the cross-section classification framework for the design of welded I-section beams failing by local buckling. Through comparisons of the flat width-to-thickness ratios against the slenderness limits for all the constituent plate elements, cross-sections in bending are classified as compact, non-compact and slender sections in AS 4100 (AS, 2016), corresponding to Class 1 (and 2), Class 3, and Class 4 sections defined in EN 1993-1-12 (CEN, 2007). Note that the AS 4100 slenderness limits between non-compact and compact plate elements and between slender and non-compact plate elements are respectively termed as the plasticity and yield slenderness limits (corresponding to the EC3 Class 2 and Class 3 slenderness limits), as presented in Table 4.14. Graphical evaluation of the AS yield slenderness limits for outstand elements in compression ($c_f/(t\epsilon_{AS})=14$), internal elements in bending ($c_w/(t\epsilon_{AS})=115$) and outstand elements in triangular compressive stress gradients ($c_f/(t\epsilon_{AS})=22$) as well as the AS plasticity slenderness limits for internal elements in bending ($c_w/(t\epsilon_{AS})=82$) and outstand elements in compression ($c_f/(t\epsilon_{AS})=8$) was carried out based on the relevant test and numerical data, and shown in Figures 4.33–4.37, respectively. The results of the graphical evaluations generally indicated that the current AS yield limits are safe but considerably conservative when used for the classification of both internal webs and outstand flanges of S690 high strength steel welded I-section beams, while the plasticity slenderness limits yield a good level of accuracy in the plate element classification.

Table 4.14: Established slenderness limits in AS 4100

Plate element type	Loading condition	Yield limit	Plasticity limit
Outstand flange	Compression	$14\epsilon_{AS}$	$8\epsilon_{AS}$
Internal web	Bending	$115\epsilon_{AS}$	$82\epsilon_{AS}$
Outstand flange	Compressive stress gradient	$22\epsilon_{AS}$	—

Figure 4.33: AS yield slenderness limit for outstand elements in compression

Figure 4.34: AS yield slenderness limit for internal elements in bending

Figure 4.35: AS yield slenderness limit for outstand elements under triangular compressive stress gradients

Figure 4.36: AS plasticity slenderness limit for internal elements in bending

Figure 4.37: AS plasticity slenderness limit for outstand elements in compression

The current AS 4100 (AS, 2016) adopts M_{pl} as the design bending moment resistances for compact welded I-section beams, but with an upper limit of $1.5M_{el}$, as given by Equation (4.5), and takes into due account partial plasticity in the predictions of cross-section bending moment resistances for non-compact welded I-section beams, with the design formulation shown by Equation (4.6), where M_c is taken as the minimum of M_{pl} and $1.5M_{el}$ of the examined non-compact I-section, λ_s is equal to the c/t ratio of the most slender constituent plate element of the non-compact I-section in bending; note that the most slender constituent plate element is defined as the element with the greatest plate element c/t to yield slenderness limit ratio – see Table 4.14, and λ_{sy} and λ_{sp} are the corresponding yield and plasticity limits for the most slender constituent plate element and presented in Table 4.14. Regarding slender welded I-section beams, the effective bending moment resistances, specified in AS 4100 (AS, 2016), are determined through multiplying the cross-section elastic moment capacities M_{el} by

the reduction factor ρ_{AS} , as given by Equation (4.7); note that the reduction factor ρ_{AS} is calculated as (λ_{sy}/λ_s) and $(\lambda_{sy}/\lambda_s)^2$ for welded I-sections with the most slender plate element being subjected to uniform compression (i.e. major-axis bending case) and stress gradients (i.e. minor-axis bending case), respectively.

$$M_{u,AS} = M_{pl} \leq 1.5M_{el} \quad (4.5)$$

$$M_{u,AS} = M_{el} + (M_c - M_{el}) \left(\frac{\lambda_{sy} - \lambda_s}{\lambda_{sy} - \lambda_{sp}} \right) \quad (4.6)$$

$$M_{u,AS} = \rho_{AS} M_{el} \quad (4.7)$$

The accuracy of the current Australian standard AS 4100 (CEN, 2016) for the design of S690 high strength steel welded I-section beams failing by local buckling was assessed by comparing the obtained test and numerical failure bending moments with the AS predicted cross-section bending moment resistances. As evident in Figure 4.38 and Table 4.15, the current AS 4100 (CEN, 2016) yields accurate and consistent cross-section bending moment resistance predictions for S690 high strength steel compact welded I-section beams in both major-axis bending and minor-axis bending and S690 high strength steel non-compact welded I-section beams in major-axis bending, but excessively underestimates the cross-section bending moment resistances for S690 high strength steel non-compact welded I-section beams bent about the minor principal axis and S690 high strength steel slender welded I-section beams in both major-axis bending and minor-axis bending.

Figure 4.38: Comparison of test and numerical failure moments with AS bending moment resistance predictions

Table 4.15: Comparisons of test and numerical failure moments against AS predicted bending moment resistances

Axis of bending	Class of cross-section	No. of test data	No. of FE data	$M_u / M_{u,AS}$	
				Mean	COV
Major axis	Compact I-section	2	21	1.08	0.01
	Non-compact I-section	4	80	1.10	0.03
	Slender I-section	0	84	1.58	0.18
	Sub-total	6	185	1.31	0.24
Minor axis	Compact I-section	2	8	1.17	0.03
	Non-compact I-section	4	51	1.33	0.05
	Slender I-section	0	42	3.22	0.45
	Sub-total	6	101	2.05	0.67
Total		12	286	1.57	0.92

4.4.3.3 American specification ANSI/AISC 360-16

The existing American specification ANSI/AISC 360-16 (AISC, 2016), with the scope of application covering steels with grades up to S690, also adopts the cross-section classification framework for the design of welded I-section beams prone to local instability. Through comparisons of the flat width-to-thickness ratios against the corresponding slenderness limits for all the constituent plate elements, the existing ANSI/AISC 360-16 (AISC, 2016) classifies cross-sections in bending as slender, non-compact and compact sections, similar to the Australian standard AS 4100 (AS, 2016); note that the flat element widths of flanges of I-shaped sections are defined as half of the full flange widths in ANSI/AISC 360-16 (AISC, 2016), while the flat element widths of webs are defined as the clear distances between the flanges. The American specification ANSI/AISC 360-16 (AISC, 2016) terms the slenderness limits for slender/non-compact and non-compact/compact plate elements as non-compact and compact limiting width-to-thickness ratios, as summarised in Table 4.16. The test and numerical results on S690 high strength steel welded I-section beams were adopted to evaluate the accuracy of the AISC non-compact limiting width-to-thickness ratios for outstand elements in compression ($c_f/(t\epsilon_{AISC}k_c^{0.5})=1.14$), internal elements in bending ($c_w/(t\epsilon_{AISC})=5.7$) and outstand elements in triangular compressive stress gradients ($c_f/(t\epsilon_{AISC})=1.0$) as well as the compact limiting width-to-thickness ratios for internal elements in bending ($c_w/(t\epsilon_{AISC})=3.76$) and outstand elements in compression ($c_f/(t\epsilon_{AISC})=0.38$), with the results of the respective graphical evaluations shown in Figures 4.39–4.43. It was found that that all the AISC limiting width-to-thickness ratios are safe and generally accurate when used for the classification of plate elements of S690 high strength steel welded I-section beams, except for the non-compact limiting width-to-thickness ratio for outstand elements in compression, which results in unsafe plate element classification.

Table 4.16: Established slenderness limits in ANSI/AISC 360-16

Plate element type	Loading condition	Non-compact limiting width-to-thickness ratio	Compact limiting width-to-thickness ratio
Outstand flange	Compression	$1.14\epsilon_{AISC}k_c^{0.5}$	$0.38\epsilon_{AISC}$
Internal web	Bending	$5.7\epsilon_{AISC}$	$3.76\epsilon_{AISC}$
Outstand flange	Compressive stress gradient	$1.0\epsilon_{AISC}$	—

Figure 4.39: AISC non-compact limiting width-to-thickness ratio for outstand elements in compression

Figure 4.40: AISC non-compact limiting width-to-thickness ratio for internal elements in bending

Figure 4.41: AISC non-compact limiting width-to-thickness ratio for outstand elements under triangular compressive stress gradients

Figure 4.42: AISC compact limiting width-to-thickness ratio for internal elements in bending

Figure 4.43: AISC compact limiting width-to-thickness ratio for outstand elements in compression

With regard to welded I-section beams bent about the cross-section major principal axis, ANSI/AISC 360-16 (AISC, 360) prescribes the use of plastic moment capacities (M_{pl}) as the design bending moment resistances for both compact I-sections and non-compact I-sections with compact outstand flanges and non-compact internal webs, and takes into account partial plasticity in the predictions of bending moment resistances for non-compact I-sections with non-compact outstand flanges and compact or non-compact internal webs, as given by Equation (4.8), where λ_{pf} and λ_{rf} are the compact and non-compact limiting width-to-thickness ratios for outstand flanges under compression, respectively, as reported in Table 4.16. The AISC design cross-section effective (reduced) bending moment capacities for slender I-section beams with (i) slender webs and compact flanges, (ii) slender webs and non-compact flanges and (iii) slender flanges are calculated from Equations (4.9)–(4.11), respectively. Regarding welded I-section beams subjected to minor-axis bending, the current ANSI/AISC 360-16 (AISC, 2016) specifies the design cross-section bending moment resistances as the minimum of M_{pl} and $1.6M_{el}$ for compact I-sections, while the design cross-section bending moment resistances of non-compact and slender I-sections are determined according to Equations (4.8) and (4.12), respectively.

$$M_{u,AISC} = M_{pl} - \left(M_{pl} - 0.7M_{el} \right) \left(\frac{c_f/t - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \quad (4.8)$$

$$M_{u,AISC} = M_{eff,AISC} = M_{el} \left[1 - \frac{c_w/(2c_f)}{1200 + 300(c_w/(2c_f))} (c_w/t - 5.7\epsilon_{AISC}) \right] \leq M_{el} \quad (4.9)$$

$$M_{u,AISC} = M_{eff,AISC} = M_{el} \left[1 - 0.3 \left(\frac{c_f/t - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \leq M_{el} \quad (4.10)$$

$$M_{u,AISC} = M_{eff,AISC} = M_{el} \frac{0.9k_c}{(c_f/t\epsilon_{AISC})^2} \leq M_{el} \quad (4.11)$$

$$M_{u,AISC} = M_{eff,AISC} = M_{el} \frac{0.69}{(c_f/t\epsilon_{AISC})^2} \leq M_{el} \quad (4.12)$$

Graphical and quantitative evaluations of the AISC design cross-section bending moment resistances were performed based on the test and numerical failure bending moments of the considered S690 high strength steel welded I-section beams, with the results shown in Figure 4.44 and Table 4.17, respectively. It is evident that the test and numerical failure bending moments of S690 high strength steel compact welded I-section beams (in bending about both the major and minor principal axes) and S690 high strength steel non-compact welded I-section beams bent about the major principal axis are well predicted by the current ANSI/AISC 360-16 (AISC, 2016), while the design cross-section bending moment resistance predictions of S690 high strength steel non-compact welded I-section beams in minor-axis bending and S690 high strength steel slender welded I-section beams in both minor-axis and major-axis bending, determined from ANSI/AISC 360-16 (AISC, 2016), are excessively conservative and scattered.

Figure 4.44: Comparison of test and numerical failure moments with AISC bending moment resistance predictions

Table 4.17: Comparisons of test and numerical failure moments against AISC predicted bending moment resistances

Axis of bending	Class of cross-section	No. of test data	No. of FE data	$M_u/M_{u,AISC}$	
				Mean	COV
Major axis	Compact I-section	2	36	1.06	0.02
	Non-compact I-section	4	131	1.11	0.06
	Slender I-section	0	18	1.35	0.11
	Sub-total	6	185	1.12	0.09
Minor axis	Compact I-section	2	28	1.09	0.03
	Non-compact I-section	4	40	1.30	0.12
	Slender I-section	0	33	3.13	0.35
	Sub-total	6	101	1.81	0.61
Total		12	286	1.36	0.74

4.5 Concluding remarks

An in-depth testing and numerical modelling programme has been carried out to investigate the material properties, membrane residual stresses and load-carrying capacities of S690 high strength steel welded I-section stub columns (in compression) and beams (in bending). The testing programme was performed on eight S690 high strength steel welded I-sections, and included material tensile coupon tests, membrane residual stress measurements, initial local geometric imperfection measurements, sixteen concentrically loaded stub column tests, and twelve in-plane four-point bending tests, with six for each axis of bending. A new predictive model for membrane residual stress distributions and amplitudes in S690 high strength steel welded I-sections was proposed. In conjunction with the testing programme, a numerical modelling programme was also carried out, including a validation study, where FE models were developed and validated against the test results, and a parametric study, where the validated FE models were employed to generate further numerical data over a wide range of cross-section aspect ratios and dimensions. The obtained test and numerical results were carefully analysed and then employed to

evaluate the accuracy of the slenderness limits for classifications of S690 high strength steel internal and outstand plate elements and the local buckling design rules for S690 high strength steel welded I-section stub columns and beams, as given in EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016).

The results of the evaluation of the existing design rules for S690 high strength steel welded I-section stub columns generally indicated that (i) all the three sets of codified slenderness limits between slender and non-slender outstand (or internal) plate elements are safe and accurate when applied to S690 high strength steel welded I-sections in compression, and (ii) all the three design standards generally yield accurate and consistent cross-section compression resistance predictions for both S690 high strength steel non-slender and slender welded I-section stub columns. Regarding the evaluation of the existing design rules for S690 high strength steel welded I-section beams, it was generally found that (i) all the established codified slenderness limits are safe for the classifications of plate elements and cross-sections of S690 high strength steel welded I-section beams, with an exception being the AISC non-compact limiting width-to-thickness ratio for outstand elements in compression, which was found to yield unsafe plate element classification, and (ii) the codified local buckling design rules in all the three design standards were shown to yield accurate and consistent cross-section bending moment resistance predictions for S690 high strength steel compact (Class 1 and 2) welded I-section beams in bending about both the principal axes and S690 high strength steel non-compact (Class 3) welded I-sections bent about the major principal axis, while the predicted cross-section bending resistances, determined from all the three design standards, are unduly conservative S690 high strength steel for non-compact (Class 3) welded I-sections in minor-axis bending and S690 high strength steel slender (Class 4) welded I-section beams in both major-axis bending and minor-axis bending.

CHAPTER 5

LOCAL STABILITY OF S690 HIGH STRENGTH STEEL WELDED I-SECTIONS UNDER COMBINED LOADING

5.1 Introduction

Chapter 4 experimentally and numerically investigated the local buckling behaviour of S690 high strength steel welded I-sections under isolated loading, while the cross-section resistances and local stability of S690 high strength steel welded I-sections subjected to combined compression and bending moment were studied through a testing and numerical modelling programme in this chapter. The testing programme was firstly conducted, including initial local geometric imperfection measurements and ten major-axis and minor-axis combined loading tests. In conjunction with the testing programme, a numerical modelling programme was performed, where finite element models were developed and validated against the obtained test results, and then used to perform parametric studies to derive further numerical data. The test and numerical data were analysed and used to evaluate the design interaction curves, as given in the European code EN 1993-1-12 (CEN, 2007), American specification ANSI/AISC 360-16 (AISC, 2016) and Australian standard AS 4100 (AS, 2016), for S690 high strength steel welded I-sections under combined loading. The findings of this research work have been reported in Sun et al. (2021c).

5.2 Testing

5.2.1 General

Given that there have been no experimental investigations into S690 high strength steel welded I-sections under combined compression and bending moment, a testing programme was firstly conducted. Ten S690 high strength steel welded I-section stub column specimens with the same cross-section size I-80×60×5 were employed in this testing programme, with five tested under combined loading about each principal axis. The adopted I-section I-80×60×5 is classified as Class 2, based on the Class 2 slenderness limits set out in EN 1993-1-12 (CEN, 2007), and also categorised as a compact section, according to the plasticity slenderness limits given in AS 4100 (AS, 2016) and the limiting width-to-thickness ratios set out in ANSI/AISC 360-16 (AISC, 2016). It is worth noting that the material properties and membrane residual stresses of the studied S690 high strength steel welded I-sections have been experimentally measured and fully reported in Chapter 4. The overall testing programme included ten combined loading tests and supplementary initial local geometric imperfection measurements.

5.2.2 Combined loading tests

Eccentric compression (combined loading) tests were conducted on ten S690 high strength steel welded I-section stub column specimens to study their cross-section behaviour and resistances under combined compression and bending moment. Five eccentric compression tests were performed about the major principal axis, while the other five were conducted about the minor principal axis. The initial loading eccentricities were varied between 5 mm and 60 mm, resulting in a wide range of loading combinations (i.e. compression load-to-bending moment ratios) being considered. The geometric dimensions and initial local geometric imperfection of

each specimen were measured before two 15 mm thick end plates were welded to its end sections, as reported in Table 5.1; Figure 5.1 illustrates the notations of the cross-section geometric parameters for welded I-sections. Note that the test setup and procedures used for the initial local geometric imperfection measurements were the same as those detailed in Section 3.2.4. The identifier of each specimen comprises two letters ‘MA’ or ‘MI’ (signifying major-axis or minor-axis combined loading) and a number (differentiating specimens with different initial loading eccentricities), e.g., MA-1.

Table 5.1: Measured geometric properties of S690 high strength steel welded I-section stub column specimens under combined loading

Axis of combined loading	Specimen ID	L (mm)	h (mm)	b_f (mm)	t (mm)	ω_0 (mm)
Major axis	MA-1	240	79.52	59.47	4.98	0.06
	MA-2	239	79.22	59.23	5.01	0.10
	MA-3	240	79.47	60.00	4.98	0.06
	MA-4	240	79.60	59.88	4.97	0.11
	MA-5	238	79.55	59.79	5.00	0.08
Minor axis	MI-1	238	79.95	59.22	4.99	0.08
	MI-2	240	80.04	59.07	4.97	0.05
	MI-3	237	79.90	60.01	4.97	0.07
	MI-4	238	78.97	59.45	4.95	0.10
	MI-5	238	79.50	59.88	4.97	0.10

Figure 5.1: Notations of welded I-section and locations of strain gauges attached to stub column specimens

An INSTRON 5000 kN testing machine was employed to perform all the combined loading tests through applying eccentric compression loads onto the specimen ends, at a constant loading rate of 0.3 mm/min. The testing machine was equipped with knife-edge devices, which offered pin-ended boundary conditions to the specimen ends. The knife-edge device consists of a pit plate with a semi-circular groove and a wedge plate containing a knife-edge wedge, as depicted in Figure 5.2. Prior to testing,

each specimen was placed between the top and bottom wedge plates, and their relative position, monitored by a laser and two spirit levels, was carefully adjusted, in order to achieve proper member alignment and pre-specified initial loading eccentricity. Upon completion of the member alignment and position adjustment, the specimen was anchored through bolting the end plates to the wedge plates.

(a) Major-axis combined loading

(b) Minor-axis combined loading

Figure 5.2: Combined loading test setups

The setup for the combined loading tests about the major principal axis is depicted in Figure 5.2(a), including one LVDT, attached to the flange of the specimen at mid-height to measure the lateral deflection to the major principal axis, and two strain gauges, attached to the outer faces of the flange-to-web junctions at mid-height – see Figure 5.1(a), to record the longitudinal strains at these positions. A similar setup was used for the minor-axis combined loading tests, as shown in Figure 5.2(b), but with one LVDT attached to the mid-height of the web and two pairs of strain gauges attached to the flanges at mid-height – see Figure 5.1(b). The readings from the LVDT and strain gauges were used to calculate the actual initial loading eccentricity e_0 of each specimen, according to Equation (5.1) (Zhao et al., 2016b; Liang et al., 2019), where I is the gross second moment of area about the axis of combined loading, N is the applied compression load, $\varepsilon_{max}-\varepsilon_{min}$ is the difference of the strains recorded by the two strain gauges for the major-axis combined loading tests (or the two pairs of strain gauges for the minor-axis combined loading tests), Δ is the lateral deflection at the specimen mid-height recorded by the LVDT, and D is the distance between the two (or two pairs of) stain gauges; $D=h$ for the combined loading tests about the major principal axis – see Figure 5.1(a), while $D=b_f-2d_s$ for the combined loading tests about the minor principal axis – see Figure 5.1(b). Note that Equation (5.1) was derived by assuming that the structural response remained elastic; it was therefore recommended that no more than 15% of the predicted ultimate load be used to calculate e_0 (Zhao et al., 2016b).

$$e_0 = \frac{EI(\varepsilon_{max} - \varepsilon_{min})}{ND} - \Delta \quad (5.1)$$

The failure modes of the S690 high strength steel welded I-section stub column specimens under major-axis and minor-axis combined loading are displayed in Figures 5.3(a) and 5.3(b), respectively. Figures 5.4(a) and 5.4(b) respectively present the full load–mid-height lateral deflection curves for the specimens under major-axis and minor-axis combined loading, with the key test results listed in Table 5.2, where

e_0 is the actual initial loading eccentricity, as calculated from Equation (5.1), N_u is the failure load, Δ_u is the mid-height lateral deflection corresponding to the failure load, and $M_u = N_u(e_0 + \Delta_u)$ is the mid-height failure moment.

(a) Failed stub column specimens under major-axis combined loading (from left to right: MA-1, MA-2, MA-3, MA-4 and MA-5)

(b) Failed stub column specimens under major-axis combined loading (from left to right: MI-1, MI-2, MI-3, MI-4 and MI-5)

Figure 5.3: Failure modes of S690 high strength steel welded I-section stub column specimens under combined loading

Table 5.2: Key test results for S690 high strength steel welded I-section stub column specimens under combined loading

Axis of combined loading	Specimen ID	e_0 (mm)	N_u (kN)	Δ_u (mm)	M_u (kNm)
Major axis	MA-1	4.9	607.0	1.1	3.6
	MA-2	11.8	505.4	2.4	7.2
	MA-3	19.3	454.9	3.0	10.1
	MA-4	34.8	358.7	5.4	14.4
	MA-5	59.1	259.1	6.3	17.0
Minor axis	MI-1	4.8	511.5	3.7	4.3
	MI-2	10.0	390.9	5.7	6.2
	MI-3	19.6	260.8	6.9	6.9
	MI-4	36.5	159.9	7.3	7.0
	MI-5	61.5	101.0	8.0	7.0

(a) Major-axis combined loading

(b) Minor-axis combined loading

Figure 5.4: Load–mid-height lateral deflection curves for S690 high strength steel welded I-section stub column specimens under combined loading

5.3 Numerical modelling

5.3.1 General

In conjunction with the testing programme, a numerical modelling programme, was conducted using the finite element analysis software ABAQUS (2014). A validation study was firstly performed where FE models were developed and validated against the test results. The validated FE models were then adopted in the parametric studies to generate further numerical data over a wide range of cross-section dimensions and loading combinations.

5.3.2 Development and validation of finite element models

For the development of FE models for S690 high strength steel welded I-section stub columns under combined loading, the procedures and techniques of the selection of mesh size, modelling of material properties and incorporation of membrane residual stresses and initial local geometric imperfections were the same as those described in Section 3.3.2. Four local imperfection magnitudes, including the measured value ω_0 and three fractions of the plate thickness ($t/100$, $t/30$ and $t/10$), were used to factor the initial local geometric imperfection pattern, in order to evaluate the influence of the imperfection magnitudes on the structural behaviour of S690 high strength steel welded I-sections under combined loading. Regarding the modelling of boundary conditions, the two end sections of each FE model were firstly coupled to two reference points, which were located eccentrically to the cross-section principal axis, with the eccentricity taken as the corresponding measured value e_0 – see Table 5.2, and also offset longitudinally from the end sections by 70 mm, i.e. the distance from the rotation centre of the knife-edge to the specimen end in the experiments – see Figures 5.2(a) and 5.2(b). Then, one reference point was allowed to translate along

the longitudinal axis and rotation about the axis of combined loading, while the other reference point was only allowed to rotate about the same axis.

Upon development of the eccentrically loaded S690 high strength steel welded I-section stub column FE models, the modified Riks method (ABAQUS, 2014) was used to analyse the developed FE models and derive the numerical failure loads, load–mid-height lateral deflection curves and failure modes. The accuracy of the developed FE models was evaluated through comparisons of the numerically derived results against their experimental counterparts. Table 5.3 presents the ratios of numerical to test failure loads for the tested specimens, with the results generally revealing that (i) the test failure loads are well predicted for all the four employed local imperfection magnitudes, and (ii) the best agreement is achieved when the local imperfection magnitude is taken as $t/100$. The experimental and numerical failure modes for two typical specimens MA-1 (under major-axis combined loading) and MI-4 (under minor-axis combined loading) are compared in Figures 5.5 and 5.6, respectively, showing excellent agreement. Figures 5.7 and 5.8 depict the test and numerical load–mid-height lateral deflection curves of the two aforementioned specimens, where the test load–mid-height lateral deflection curves are well captured by their numerical counterparts. To conclude, the developed FE models are capable of accurately simulating the major-axis and minor-axis combined loading tests on S690 high strength steel welded I-section stub columns, and thus considered to be validated.

Table 5.3: Comparison of FE failure loads with test failure loads for varying local imperfection magnitudes

Axis of combined loading	Specimen ID	Finite element N_u / Test N_u			
		ω_0	$t/100$	$t/30$	$t/10$
Major axis	MA-1	0.979	0.980	0.975	0.966
	MA-2	0.994	1.008	0.997	0.971
	MA-3	0.972	0.974	0.950	0.935
	MA-4	0.961	0.963	0.956	0.947
	MA-5	1.003	1.006	0.975	0.945
Minor axis	MI-1	0.984	0.986	0.976	0.956
	MI-2	0.944	0.944	0.939	0.935
	MI-3	0.968	0.969	0.960	0.940
	MI-4	0.941	0.953	0.938	0.930
	MI-5	0.944	0.964	0.939	0.926
	Mean	0.969	0.974	0.961	0.945
	COV	0.022	0.021	0.020	0.017

Figure 5.5: Test and FE failure modes for specimen MA-1 **Figure 5.6:** Test and FE failure modes for specimen MI-4

Figure 5.7: Test and FE load–mid-height lateral deflection curves for specimen MA-1 under major-axis combined loading

Figure 5.8: Test and FE load–mid-height lateral deflection curves for specimen MI-4 under minor-axis combined loading

5.3.3 Parametric studies

Upon validation of the eccentrically loaded stub column FE models, parametric studies were performed in this section to expand the test data pool over a wider range of cross-section dimensions and loading combinations. In the present parametric studies, the modelled I-sections comprised all three non-slender classes (i.e. Class 1, 2 and 3) of cross-sections, based on the slenderness limits given in EN 1993-1-12 (CEN, 2007), and also categorised as non-slender cross-sections in accordance with the slenderness limits set out in AS 4100 (AS, 2016) and the limiting width-to-thickness ratios given in ANSI/AISC 360-16 (AISC, 2016). Specifically, the outer section heights of the modelled I-sections were kept at 100 mm, and the flange widths were ranged from 50 mm to 100 mm, with the plate thicknesses varied between 4 mm and 12 mm, resulting in a broad range of cross-section aspect ratios and dimensions being considered. The length of each FE model was equal to three times the outer section height (i.e. 300 mm). Moreover, the initial loading eccentricities were varied between 1 mm and 1000 mm, leading to an extensive range of loading combinations being accounted for. The geometric dimensions, initial loading eccentricities and load-carrying capacities of the modelled S690 high strength steel welded I-section stub columns under combined loading are reported in Appendix. The initial local geometric imperfection magnitudes were taken as $t/100$. A total of 150 numerical data on S690 high strength steel welded I-section stub columns under combined loading were generated from the parametric study, with 75 for combined loading about each principal axis.

5.4 Evaluation of existing design standards

5.4.1 General

In this section, the existing design rules, as set out in EN 1993-1-12 (CEN, 2007), AS 4100 (AS, 2016) and ANSI/AISC 360-16 (AISC, 2016), for S690 high strength steel welded I-sections under combined compression and bending moment, were firstly presented and then evaluated through comparisons of the test and numerical failure loads N_u with the unfactored design failure loads $N_{u,pred}$. Tables 5.4–5.6 respectively report the mean $N_u/N_{u,pred}$ ratios and the corresponding COVs for the three considered design standards.

5.4.2 Eurocode EN 1993-1-12

The current European code EN 1993-1-12 (CEN, 2007) for high strength steels with grades greater than S460 up to S700 adopts a linear elastic interaction formula for the design of Class 3 welded I-sections under combined compression and bending moment, which is a simple linear summation of the utilisation ratio for each stress resultant, with the maximum attainable stress (at failure) within the cross-section limited to the material yield stress f_y – see Equation (5.2), where $N_{u,pred}$ is the design axial compression load at failure, $M_{u,y}$ and $M_{u,z}$ are the design failure moments about the major and minor principal axes, respectively; note that $M_{u,y}$ (or $M_{u,z}$) is taken as $N_{u,pred}$ multiplied by the summation of e_0 and Δ_u , $M_{el,y}=W_{el,y}f_y$ and $M_{el,z}=W_{el,z}f_y$ are respectively the cross-section elastic moment capacities about the major and minor principal axes, respectively, where $W_{el,y}$ and $W_{el,z}$ are the corresponding elastic section moduli.

$$\frac{N_{u,pred}}{Af_y} + \frac{M_{u,y}}{M_{el,y}} + \frac{M_{u,z}}{M_{el,z}} \leq 1 \quad (5.2)$$

Regarding Class 1 and 2 welded I-sections under combined compression and bending moment, nonlinear design interaction formulae, as derived based on full plasticity throughout the whole cross-section, are employed in EN 1993-1-12 (CEN, 2007). The corresponding nonlinear design interaction formulae for the major-axis and minor-axis combined loading cases are given by Equations (5.3) and (5.4), where $M_{pl,y}$ and $M_{pl,z}$ are the cross-section plastic moment capacities about the major and minor principal axes, respectively, defined as the products of the material yield stress f_y and the major-axis and minor-axis plastic section moduli ($W_{pl,y}$ and $W_{pl,z}$), $n=N_{u,pred}/(Af_y)$ is the ratio of the design axial compression load to the cross-section yield load, $a_w=(A-2tb_f)/A$ is the web-to-gross area ratio, with an upper limit of 0.5, and $M_{N,y}$ and $M_{N,z}$ are the reduced cross-section major-axis and minor-axis plastic moment resistances, respectively, taking into account the existence of the axial compression load $N_{u,pred}$.

$$M_{u,y} \leq M_{N,y} = M_{pl,y} \frac{(1-n)}{(1-0.5a_w)} \leq M_{pl,y} \quad (5.3)$$

$$M_{u,z} \leq M_{N,z} = \begin{cases} M_{pl,z} & \text{for } n \leq a_w \\ M_{pl,z} \left[1 - \left(\frac{n-a_w}{1-a_w} \right)^2 \right] & \text{for } n > a_w \end{cases} \quad (5.4)$$

The design interaction rules, as given in EN 1993-1-12 (CEN, 2007), for S690 high strength steel welded I-sections under combined loading were evaluated through comparisons of the test and numerical failure loads against the EC3 design failure loads. Table 5.4 presents the mean test and numerical to EC3 design failure load ratios and the corresponding COVs for various classes of S690 high strength steel non-slender welded I-sections under combined loading. Figures 5.9(a) and 5.9(b) respectively depict the graphical evaluation results for the major-axis and minor-axis combined loading cases, where the experimentally and numerically obtained failure bending moments and axial loads are normalised by the cross-section plastic moment

resistances and yield loads, and compared against the average EC3 linear and nonlinear design interaction curves, as defined by Equation (5.2) and Equations (5.3) and (5.4), respectively; note that the EC3 design interaction curves were averaged, owing to the variations in the values of a_w , $M_{el,z}/M_{pl,z}$ and $M_{el,y}/M_{pl,y}$ for I-sections with different cross-section sizes. Both the quantitative and graphical evaluations indicated that (i) the existing EN 1993-1-12 (CEN, 2007) yields safe, accurate and consistent resistance predictions for S690 high strength steel Class 1 (and 2) welded I-sections under both major-axis and minor-axis combined loading, (ii) the predicted resistances are slightly conservative though safe for S690 high strength steel Class 3 welded I-sections under combined loading about the major principal axis, and (iii) the load-carrying capacities of S690 high strength steel Class 3 welded I-sections under minor-axis combined loading are overly underestimated by EN 1993-1-12 (CEN, 2007).

Table 5.4: Comparisons of test and FE failure loads with EN 1993-1-12 resistance predictions

Axis of combined loading	Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,pred}$	
				Mean	COV
Major axis	Class 1 and 2	5	50	1.12	0.04
	Class 3	0	25	1.25	0.07
	Sub-total	5	75	1.16	0.08
Minor axis	Class 1 and 2	5	50	1.06	0.04
	Class 3	0	25	1.79	0.09
	Sub-total	5	75	1.29	0.27
Total		10	150	1.22	0.22

(a) Major-axis combined loading

(a) Minor-axis combined loading

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

5.4.3 Australian standard AS 4100

The current Australian standard AS 4100 (AS, 2016), with the scope of application covering steels with grades up to S690, employs a linear elastic interaction formula for the design of both compact and non-compact welded I-sections under combined

loading, as defined by Equation (5.5), where $M_{s,y}$ and $M_{s,z}$ are the AS cross-section bending moment resistances about the major and minor principal axes, respectively. Note that the AS linear interaction formula (i.e. Equation (5.5)) is the same as its EC3 counterpart (i.e. Equation (5.2)), with the only difference being the calculation of the cross-section bending moment resistances. AS 4100 (AS, 2016) adopts the plastic moment capacities as the cross-section bending resistances for compact welded I-sections, but with an upper limit of 1.5 times the corresponding elastic moment resistances, and takes into account partial plasticity in the predictions of cross-section bending moment resistances for non-compact welded I-sections, with the design formulation given in Clause 5.2.4 of AS 4100 (AS, 2016) and presented in Section 4.4.3.2, thus the resulting AS cross-section bending resistances are greater than the corresponding elastic moment capacities.

$$\frac{N_{u,pred}}{Af_y} + \frac{M_{u,y}}{M_{s,y}} + \frac{M_{u,z}}{M_{s,z}} \leq 1 \quad (5.5)$$

The accuracy of the AS design rules for S690 high strength steel welded I-sections under combined loading was evaluated. The experimentally and numerically obtained failure moments and loads were normalised by the cross-section plastic moment capacities and yield loads, compared against the average AS linear design interaction curves, and depicted in Figures 5.9(a) and 5.9(b) for the major-axis and minor-axis combined loading cases, respectively. It is evident that the AS linear design interaction curves lie far below the test and numerical data points, indicating excessive conservatism; this is also shown in Table 5.5, where the mean ratios of $N_u/N_{u,pred}$, together with the corresponding COVs, for S690 high strength steel non-slender (compact and non-compact) welded I-sections under major-axis and minor-axis combined loading are reported. Based on the quantitative and graphical evaluations, it was also found that the AS design interaction curve yields more conservative and scattered resistance predictions for S690 high strength steel compact welded I-sections under combined loading than the EC3 design interaction curve,

which can be attributed to the adoption of a less accurate linear interaction. For S690 high strength steel non-compact welded I-sections, the AS design interaction curve is anchored to more accurate bending end point, and thus offers a higher degree of design accuracy and consistency than the EC3 design interaction curve.

Table 5.5: Comparisons of test and FE failure loads with AS 4100 resistance predictions

Axis of combined loading	Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,pred}$	
				Mean	COV
Major axis	Compact	5	40	1.24	0.06
	Non-compact	0	35	1.22	0.06
	Sub-total	5	75	1.23	0.06
Minor axis	Compact	5	45	1.45	0.09
	Non-compact	0	30	1.45	0.08
	Sub-total	5	75	1.45	0.08
Total		10	150	1.34	0.11

5.4.4 American specification ANSI/AISC 360-16

A bi-linear interaction curve, as defined by Equation (5.6), is employed in the existing ANSI/AISC 360-16 (AISC, 2016) for the design of non-slender (compact and non-compact) welded I-sections under combined loading, where M_{cy} and M_{cz} are the AISC cross-section bending moment resistances about the major and minor principal axes, respectively. For compact welded I-sections, ANSI/AISC 360-16 (AISC, 2016) adopts $M_{pl,y}$ as the cross-section bending moment resistances about the major axis, but the minimum of $M_{pl,z}$ and $1.6M_{el,z}$ as the cross-section bending moment resistances about the minor axis. For non-compact welded I-sections, the American specification (AISC, 2016) considers partial plasticity in the determination of M_{cy} and M_{cz} , with the corresponding design formulae given in Clauses F3 and F6 of ANSI/AISC 360-16 (AISC, 2016) and presented in Section 4.4.3.3; the resulting M_{cy} and M_{cz} are therefore generally greater than the corresponding elastic moment capacities. Detailed discussions on the EC3, AS and AISC cross-section bending

moment resistances for S690 high strength steel welded I-sections have been presented in Chapter 4.

$$\begin{cases} \frac{N_{u,pred}}{Af_y} + \frac{8}{9} \left(\frac{M_{u,y}}{M_{cy}} + \frac{M_{u,z}}{M_{cz}} \right) \leq 1 & \text{for } \frac{N_{u,pred}}{Af_y} \geq 0.2 \\ \frac{N_{u,pred}}{2Af_y} + \left(\frac{M_{u,y}}{M_{cy}} + \frac{M_{u,z}}{M_{cz}} \right) \leq 1 & \text{for } \frac{N_{u,pred}}{Af_y} < 0.2 \end{cases} \quad (5.6)$$

The AISC design failure loads for S690 high strength steel non-slender welded I-sections under combined compression and bending moment were evaluated through comparisons against the test and numerical failure loads, with the results shown in Table 5.6 and Figure 5.9. The results of the comparisons generally revealed that ANSI/AISC 360-16 (AISC, 2016) results in accurate and consistent resistance predictions for S690 high strength steel non-slender welded I-sections under combined compression and major-axis bending moment, but underestimates the load-carrying capacities of those under minor-axis combined loading. In comparison with EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) leads to more conservative and scattered resistance predictions for S690 high strength steel compact welded I-sections under combined compression and bending moment, owing principally to the employment of a less accurate design interaction curve. Note that ANSI/AISC 360-16 (AISC, 2016) yields the most accurate resistance predictions for S690 high strength steel non-compact welded I-sections under combined loading among the three considered design standards (CEN, 2007; AS, 2016; AISC, 2016), due to the use of a nonlinear interaction curve anchored to accurate bending end point.

Table 5.6: Comparisons of test and FE failure loads with ANSI/AISC 360-16 resistance predictions

Axis of combined loading	Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,pred}$	
				Mean	COV
Major axis	Compact	5	55	1.16	0.04
	Non-compact	0	20	1.13	0.05
	Sub-total	5	75	1.15	0.05
Minor axis	Compact	5	55	1.30	0.09
	Non-compact	0	20	1.49	0.10
	Sub-total	5	75	1.34	0.11
Total		10	150	1.25	0.12

5.5 Concluding remarks

A thorough testing and numerical modelling programme has been conducted to investigate the local stability and load-carrying capacities of S690 high strength steel welded I-sections under combined compression and bending moment. The testing programme included initial local geometric imperfection measurements and ten combined loading tests about both the major and minor principal axes. A parallel numerical modelling programme was then performed, where finite element models were developed and validated against the test results and then adopted to perform parametric studies to expand the test data pool on S690 high strength steel welded I-sections under combined loading over a wide range of cross-section dimensions and loading combinations. The obtained test and numerical results were employed to evaluate the accuracy of the design rules, as given in the current European code EN 1993-1-12 (CEN, 2007), Australian standards AS 4100 (AS, 2016) and American specification ANSI/AISC 360-16 (AISC, 2016), for S690 high strength steel welded I-sections under combined compression and bending moment. The evaluation results generally indicated that (i) EN 1993-1-12 (CEN 2007) yields safe, accurate and consistent resistance predictions for S690 high strength steel Class 1 (and Class 2) welded I-sections under both major-axis and minor-axis combined loading, but

underestimates the cross-section resistances for S690 high strength steel Class 3 welded I-sections under combined loading, (ii) the load-carrying capacities of S690 high strength steel welded I-sections under combined compression and major-axis bending moment are well predicted ANSI/AISC 360-16 (AISC, 2016), but the AISC resistance predictions are rather conservative for S690 high strength steel welded I-sections under minor-axis combined loading, and (iii) the linear interaction curves adopted in AS 4100 (AS, 2016) lead to the overall highest level of conservatism for S690 high strength steel welded I-sections under combined loading.

CHAPTER 6

BUCKLING OF HIGH-CHROMIUM STAINLESS STEEL WELDED I-SECTION COLUMNS

6.1 Introduction

Chapters 3, 4 and 5 have investigated the cross-sectional behaviour and local stability of high-performance steel welded I-sections subjected to compression, bending, and combined compression and bending, while the member stability and load-carrying capacities of welded I-section columns and beam-columns made of new EN 1.4420 high-chromium stainless steel and S690 high strength steel are studied in the present chapter and Chapter 7 and Chapter 8, respectively. This chapter presents an experimental and numerical study on the flexural and local–flexural interactive buckling behaviour of high-chromium austenitic stainless steel welded I-section columns. The testing programme adopted four different welded I-section sizes and included twenty axially loaded pin-ended column tests and supplementary initial global and local geometric imperfection measurements on the column specimens. The testing programme was accompanied by a numerical modelling programme, including a validation study, where column FE models were developed and validated against the test results, and a parametric study, where the validated column FE models were employed to generate further numerical data over a wide range of cross-section dimensions and member effective lengths. The experimentally and numerically obtained results were adopted to evaluate the applicability of the design rules for normal stainless steel welded I-section columns, as set out in the current European

code EN 1993-1-4 (CEN, 2015) and American design guide AISC 27 (AISC, 2013), to the new high-chromium stainless steel welded I-section columns failing by flexural buckling and local–flexural interactive buckling. The findings of this research work have been reported in Sun et al. (2020a, 2020b).

6.2 Testing

6.2.1 General

A testing programme was firstly conducted to (i) investigate the minor-axis flexural buckling behaviour and resistances of high-chromium stainless steel welded I-section columns with non-slender cross-sections and (ii) study the local–flexural interactive buckling behaviour and resistances of high-chromium stainless steel welded I-section columns with slender cross-sections. Four different I-sections – I-150×100×6, I-150×120×6, I-150×150×5 and I-200×100×5, fabricated by robotic GMAW from grade EN 1.4420 high-chromium stainless steel plates, were adopted in the testing programme. Specifically, I-150×100×6 and I-150×120×6 are classified as Class 1 and Class 3 (i.e. non-slender sections), respectively, in accordance with the slenderness limits set out in the existing Eurocode EN 1993-1-4 (CEN, 2015), and also categorised as non-slender sections, according to the limiting width-to-thickness ratios given in the current American design guide AISC 27 (AISC, 2013); both of I-150×150×5 and I-200×100×5 are defined as Class 4 (slender) sections, based on the slenderness limits given in EN 1993-1-4 (CEN, 2015), and also categorised as slender sections, according to the limiting width-to-thickness ratios set out in AISC 27 (AISC, 2013). It is worth noting that the material properties and membrane residual stresses of the studied high-chromium stainless steel welded I-sections have been measured and reported in Chapter 3. Overall, the testing programme included initial global and local geometric imperfection measurements and twenty pin-ended column tests.

6.2.2 Initial geometric imperfection measurements

The structural behaviour and resistances of stainless steel (welded, cold-formed and hot-rolled) members are generally affected by their initial geometric imperfections. The initial global and local geometric imperfections of the high-chromium stainless steel welded I-section column specimens were thus measured. The test setup for initial global geometric imperfection measurements is depicted in Figure 6.1, where a column specimen is placed on the flat work bench of a CNC router and a LVDT is mounted onto the arm of the CNC router. The LVDT was moved along the centreline of the internal web of the welded I-section column specimen, with the readings recorded at the two ends and mid-height. The initial mid-height global geometric imperfection of the column specimen to its minor principal axis ω_g was then taken as the deviation from a linear reference line (i.e. a straight line connecting the data points at the two ends) to the data point measured at mid-height, as reported in Table 6.1. Measurements on the initial local geometric imperfections of column specimens were significantly influenced by the existence of member initial global geometric imperfections, and therefore their initial local geometric imperfection magnitudes ω_0 were taken as those previously measured from the corresponding stub column specimens with the same cross-section sizes, as reported in Table 6.1.

Figure 6.1: Setup for initial global geometric imperfection measurements

Table 6.1: Measured geometric dimensions and initial geometric imperfection magnitudes of high-chromium stainless steel welded I-section column specimens

Cross-section	Specimen ID	L (mm)	L_e (mm)	h (mm)	b_f (mm)	t (mm)	ω_0 (mm)	ω_g (mm)
I-150×100×6	A1	799	909	149.49	99.17	6.04	0.06	0.13
	A2	949	1059	149.31	99.47	5.97	0.06	0.23
	A3	1150	1260	149.48	99.25	5.99	0.06	0.29
	A4	1449	1559	149.51	99.38	6.00	0.06	0.29
	A5	1749	1859	149.38	99.41	5.98	0.06	0.40
I-150×120×6	B1	848	958	149.34	119.18	6.01	0.07	0.16
	B2	1047	1157	149.08	119.32	6.00	0.07	0.24
	B3	1399	1509	149.38	119.22	6.03	0.07	0.31
	B4	1701	1811	149.52	119.63	6.00	0.07	0.37
	B5	1848	1958	149.08	119.48	6.00	0.07	0.42
I-150×150×5	C1	1000	1110	149.88	149.21	5.00	0.08	0.25
	C2	1346	1456	150.01	149.09	5.05	0.08	0.41
	C3	1550	1660	149.98	149.03	5.03	0.08	0.57
	C4	1900	2010	149.58	149.21	5.03	0.08	0.71
	C5	2350	2460	149.72	149.33	5.01	0.08	0.82
I-200×100×5	D1	699	809	199.82	99.23	5.02	0.14	0.18
	D2	947	1057	199.61	99.39	4.95	0.14	0.29
	D3	1150	1260	199.75	99.29	4.99	0.14	0.32
	D4	1449	1559	199.60	99.27	5.01	0.14	0.51
	D5	1748	1858	199.89	99.59	5.00	0.14	0.73

6.2.3 Pin-ended column tests

Compression tests were performed on pin-ended high-chromium stainless steel welded I-section columns to investigate their flexural buckling and local–flexural interactive buckling behaviour. For each adopted welded I-section, five column specimens with different member lengths were prepared, leading to a total of twenty column specimens examined in the tests. Table 6.1 reports the key geometric properties of the twenty high-chromium stainless steel welded I-section column specimens. All the column specimens were tested using an INSTRON 5000 kN servo-hydraulic testing machine at a constant rate of 0.4 mm/min. The testing machine was equipped with knife-edge devices, which offered pin-ended boundary conditions to the column specimen ends. The knife-edge device, as depicted in Figure 6.2, consists of a pit plate with a semi-circular groove and a wedge plate containing a knife-edge wedge. Prior to testing, a column specimen was firstly positioned between the top and bottom wedge plates, and their relative position, monitored by using a laser and two spirit levels, was adjusted such that (i) the member longitudinal axis was perpendicular to the wedge plates and also intersected with the knife-edges at right angles and (ii) the cross-section minor principal axis was parallel with the knife-edges. The column specimen was then anchored at both ends by stiffening plates (bolted to the wedge plates), before being placed between the top and bottom pit plates in the testing machine. Note that the distance from the end of the specimen to the rotation centre of the knife-edge device is equal to 55 mm and the effective member length of each column specimen is given as $L_e=L+110$ mm, as reported in Table 6.1.

Figure 6.2: Pin-ended column test setup

Figure 6.2 depicts the pin-ended column test setup, where a LVDT is attached to the web of the column specimen at mid-height to measure the lateral deflection along the buckling direction (i.e. to the minor principal axis) and two pairs of strain gauges, denoted as SG 1-a and SG 1-b as well as SG 2-a and SG 2-b, are attached to the outstand flanges at mid-height, with the detailed locations depicted in Figure 5.1(b) (but with $d_s=10$ mm), to measure the longitudinal strains at these positions. The readings from the LVDT and two pairs of strain gauges were used to derive the actual overall global geometric imperfection magnitude, defined as the sum of the initial global geometric imperfection magnitude at the member mid-height ω_g and the initial loading eccentricity at the member end e_0 , for each high-chromium stainless steel welded I-section column specimen, according to Equation (6.1) (Huang & Young,

2014; Zhao et al., 2016a; Buchanan et al., 2018),

$$e_0 + \omega_g = \frac{EI_z (\varepsilon_{max} - \varepsilon_{min})}{N(b_f - 2d_s)} - \Delta \quad (6.1)$$

where I_z is the gross second moment of area about the minor principal axis, $\varepsilon_{max}-\varepsilon_{min}$ is the difference of the longitudinal strains measured by the two pairs of strain gauges, N is the applied compression force, b_f-2d_s is the distance between the two pairs of strain gauges – see Figure 5.1(b), and Δ is the mid-height lateral deflection measured by the LVDT. Note that Equation (6.1) was derived based on the assumption that the structural behaviour was close to linear elastic, and it was thus recommended that no more than 15% of the expected failure load be used in the calculation of (ω_g+e_0) (Zhao et al., 2016a; Buchanan et al., 2018). For each column specimen, its position in the testing machine was carefully adjusted until the achievement of $(\omega_g+e_0) < L_e/1000$ (Gardner et al., 2016; Buchanan et al., 2018). Table 6.2 reports the final overall global geometric imperfection magnitude (ω_g+e_0) , together with the normalised value $(\omega_g+e_0)/L_e$, for each column specimen, showing that the largest normalised value $(\omega_g+e_0)/L_e$ from all the column specimens is equal to 1/1190.

Figures 6.3 and 6.4 display the load–mid-height lateral deflection curves for the tested high-chromium stainless steel welded I-section column specimens with non-slender and slender cross-sections, respectively, while the key obtained test results are reported in Table 6.2, including the failure load N_u and the mid-height lateral deflection at the failure load Δ_u . Regarding the failure modes of the tested high-chromium stainless steel column specimens, all the non-slender welded I-section column specimens underwent visible global deformations and failed by member flexural buckling about the minor principal axis, as shown in Figure 6.5, while all the slender welded I-section column specimens were found to exhibit noticeable lateral deflections about the minor principal axis coupled with localised deformations of flanges, indicating the occurrence of local–flexural interactive buckling; Figure 6.6

displays the deformed failure modes for typical high-chromium stainless steel slender welded I-section column specimens I-150×150×5-C1, I-150×150×5-C2 and I-150×150×5-C3. For each specimen series, the columns with longer member effective lengths generally possess lower failure loads and larger mid-height lateral deflections corresponding to the failure loads, owing to the increased influence from flexural buckling.

Table 6.2: Key test results for high-chromium stainless steel welded I-section column specimens

Cross-section	Specimen ID	$\omega_g + e_0$ (mm)	$(\omega_g + e_0)/L_e$	N_u (kN)	Δ_u (mm)
I-150×100×6	A1	0.32	1/2841	583.0	7.1
	A2	0.41	1/2583	506.7	9.2
	A3	0.50	1/2520	473.6	5.5
	A4	0.43	1/3626	379.3	5.2
	A5	0.59	1/3151	330.9	4.6
I-150×120×6	B1	0.42	1/2281	681.6	8.9
	B2	0.44	1/2630	614.0	11.3
	B3	0.63	1/2395	561.8	5.3
	B4	0.63	1/2875	465.3	4.8
	B5	0.71	1/2758	437.5	5.9
I-150×150×5	C1	0.59	1/1881	629.0	7.9
	C2	0.74	1/1968	583.9	12.2
	C3	0.87	1/1908	542.0	13.9
	C4	0.92	1/2185	464.3	16.7
	C5	1.21	1/2033	404.4	16.9
I-200×100×5	D1	0.68	1/1190	590.3	3.5
	D2	0.76	1/1391	522.8	5.2
	D3	0.81	1/1556	419.9	4.8
	D4	0.95	1/1641	360.2	5.1
	D5	1.14	1/1630	273.5	5.7

(a) I-150x100x6 column specimens

(b) I-150x120x6 column specimens

Figure 6.3: Load–mid-height lateral deflection curves for high-chromium stainless steel non-slender welded I-section column specimens

(a) I-150x150x5 column specimens

(b) I-200x100x5 column specimens

Figure 6.4: Load–mid-height lateral deflection curves for high-chromium stainless steel slender welded I-section column specimens

(a) I-150×100×6 column specimens

(b) I-150×120×6 column specimens

Figure 6.5: Minor-axis flexural buckling failure modes for high-chromium stainless steel non-slender welded I-section column specimens

Figure 6.6: Local–flexural interactive buckling failure modes for high-chromium stainless steel slender welded I-section column specimens (from left to right: I-150×150×5-C1, I-150×150×5-C2 and I-150×150×5-C3)

6.3 Numerical modelling

6.3.1 General

In this section, a numerical modelling programme was performed by using the nonlinear finite element software ABAQUS (2014). FE models were firstly developed and validated against the test results. The validated FE models were then adopted to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions and member effective lengths.

6.3.2 Development of finite element models

Each high-chromium stainless steel welded I-section column FE model was developed based on the corresponding measured cross-section dimensions and member effective length. The four-node shell element S4R (ABAQUS, 2014) has

been widely and successfully used in previous numerical modelling of stainless steel welded I-section members (Saliba & Gardner, 2013; Yuan et al., 2015; Bu & Gardner, 2018), and was also employed herein. A mesh sensitivity study was performed to seek suitable element sizes that allow for accurate incorporation of membrane residual stresses into the stainless steel welded I-section column FE models and also result in adequate computational accuracy and efficiency; a series of element sizes ranging between $0.5t$ and $3t$ were examined in the mesh sensitivity study, and an element size equal to t was finally selected. The plastic material model, which is provided in ABAQUS (2014) and accounts for isotropic hardening, was adopted to model the material response of the new grade EN 1.4420 high-chromium stainless steel. Given that the plastic material model requires the material properties to be specified in the format of true stress and true plastic strain, the (measured) engineering stress–strain curve of high-chromium stainless steel was converted into the true stress–plastic strain curve and then assigned to the column FE models. For each high-chromium stainless steel welded I-section, the membrane residual stress pattern and amplitudes were firstly determined by the predictive model presented in Section 3.2.3, and then assigned to the corresponding column FE models by using the ‘*INITIAL CONDITIONS’ command (ABAQUS, 2014); Figure 6.7 shows the membrane residual stress pattern and amplitudes incorporated into the column FE model for a typical specimen I-200×100×5-D3. With regard to the end section boundary conditions, each end section of the column FE model was coupled to a concentric reference point. The top reference point, located at the loaded end, was allowed for translation in the member longitudinal direction and rotation about the cross-section minor principal axis (i.e. the buckling axis), while the bottom reference point was fully restrained except for rotation about the same buckling axis; this mimics the pin-ended boundary conditions that were offered by the knife-edge devices in the column tests.

Figure 6.7: Typical membrane residual stress pattern and amplitudes (in MPa) in modelled high-chromium stainless steel welded I-section column I-200×100×5-D3 (positive values indicate tensile residual stresses while negative values indicate compressive residual stresses)

Given that the initial global and local geometric imperfections may greatly influence the structural behaviour of high-chromium stainless steel welded I-section columns, these two types of initial geometric imperfections were incorporated into each column FE model. The initial global and local imperfection distribution patterns were respectively taken as the lowest elastic flexural and local buckling mode shapes obtained from a prior eigenvalue buckling analysis (ABAQUS, 2014; Yuan et al., 2015; Zhao et al., 2016a; Buchanan et al., 2018). Three global geometric imperfection magnitudes, including the measured overall global geometric imperfection magnitude ($\omega_g + e_0$) and two fractions of the member effective length ($L_e/1000$ and $L_e/2000$), and three local geometric imperfection magnitudes, including the measured magnitude ω_0 and two fractions of the plate thickness ($t/10$ and $t/100$), were employed to scale the corresponding initial global and local geometric imperfection patterns for each column FE model. Five global and local geometric imperfection magnitude combinations, i.e. ($\omega_g + e_0$) and ω_0 , $L_e/1000$ and $t/10$, $L_e/1000$ and $t/100$, $L_e/2000$ and

$t/10$, and $L_e/2000$ and $t/100$, were examined to assess the sensitivity of the developed column FE models to the geometric imperfection magnitudes and seek the most appropriate geometric imperfection magnitude combination to be adopted in the parametric studies. Upon development of the high-chromium stainless steel welded I-section column FE models, a static Riks (ABAQUS, 2014) analysis, taking into account both the material nonlinearity and geometric nonlinearity, was carried out to derive the numerical failure loads, load–mid-height lateral deflection curves and failure modes.

6.3.3 Validation of finite element models

Validation of the developed column FE models was conducted by comparing the numerically derived results against their experimental counterparts. Table 6.3 reports the ratios of the numerical to test failure loads for all the twenty high-chromium stainless steel welded I-section column specimens. The comparison results indicated that (i) all the five global and local geometric imperfection magnitude combinations generally result in accurate and consistent predictions of the test failure loads and (ii) the best agreement between the test and numerical failure loads is obtained when the global and local geometric imperfection magnitudes are taken as $L_e/1000$ and $t/100$, respectively. Figures 6.8(a) and 6.8(b) present comparisons between the test and numerical load–mid-height lateral deflection curves for a typical non-slender welded I-section column specimen I-150×100×6-A5 and a typical slender welded I-section column specimen I-150×150×5-C3, respectively, where the test load–mid-height lateral deflection curves are well replicated by their numerical counterparts. The numerical flexural buckling failure mode for the non-slender welded I-section column specimen I-150×100×6-A5 was also found to display good agreement with the experimentally obtained failure mode, as shown in Figure 6.9, while the test and numerical failure modes for the slender welded I-section column specimen I-150×150×5-C3 are presented in Figure 6.10, where the test failure mode,

characterised by lateral deflection about the minor principal axis coupled with localised deformations of flanges, is well replicated by its numerical counterpart. Overall, it may be concluded that the developed FE models are capable of simulating the test responses of the high-chromium stainless steel welded I-section column specimens, and therefore deemed to be validated.

Table 6.3: Comparison of FE failure loads with test failure loads for varying global and local geometric imperfection magnitude combinations

Cross-section	Specimen ID	FE N_u / Test N_u				
		$(\omega_g + e_0)$ + ω_0	$L_e/1000$ + $t/10$	$L_e/1000$ + $t/100$	$L_e/2000$ + $t/10$	$L_e/2000$ + $t/100$
I-150×100×6	A1	1.027	1.008	1.009	1.021	1.022
	A2	1.034	1.010	1.012	1.027	1.028
	A3	1.016	0.985	0.986	1.011	1.011
	A4	0.995	0.970	0.971	0.984	0.985
	A5	1.034	1.008	1.010	1.026	1.027
I-150×120×6	B1	1.029	1.010	1.011	1.025	1.025
	B2	0.988	0.965	0.966	0.980	0.982
	B3	1.056	1.027	1.029	1.050	1.051
	B4	0.975	0.956	0.957	0.968	0.968
	B5	0.984	0.966	0.968	0.975	0.978
I-150×150×5	C1	0.984	0.966	0.969	0.985	0.988
	C2	0.992	0.970	0.975	0.991	0.994
	C3	1.052	1.030	1.034	1.052	1.057
	C4	0.996	0.978	0.982	0.990	0.994
	C5	1.021	1.001	1.004	1.019	1.023
I-200×100×5	D1	1.032	1.013	1.019	1.041	1.047
	D2	1.014	0.982	0.986	1.021	1.026
	D3	0.995	0.966	0.973	0.998	1.005
	D4	0.983	0.962	0.966	0.986	0.989
	D5	1.025	1.014	1.016	1.027	1.029
	Mean	1.012	0.989	0.993	1.009	1.011
	COV	0.024	0.024	0.024	0.025	0.026

(a) I-150×100×6-A5

(b) I-150×150×5-C3

Figure 6.8: Test and FE load–mid-height lateral deflection curves for two typical high-chromium stainless steel welded I-section column specimens

Figure 6.9: Test and FE minor-axis flexural buckling failure modes for high-chromium stainless steel non-slender welded I-section column specimen I-150×100×6-A5

Figure 6.10: Test and FE local-flexural interactive buckling failure modes for high-chromium stainless steel slender welded I-section column specimen I-150×150×5-C3

6.3.4 Parametric studies

The validated column FE models were employed to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions and member effective lengths. In the present parametric studies, the same modelling assumptions, procedures and techniques, as those described in Section 6.3.2, were employed, but with the initial global and local imperfection magnitudes taken as $L_e/1000$ and $t/100$, respectively. Regarding the cross-section geometric dimensions, the outer section heights of the modelled non-slender and slender welded I-sections were respectively fixed at 100 mm and 200 mm, and the flange widths were varied between 50 mm and 200 mm, with the plate thicknesses set to be equal and ranging from 3 mm to 9.5 mm, which result in a broad range of cross-section aspect ratios and dimensions being considered. It is worth noting that the modelled I-sections cover all four classes (i.e. Class 1, Class 2, Class 3 and Class 4) of cross-sections in accordance with the slenderness limits set out in EN 1993-1-4 (CEN, 2015). The effective lengths of the column FE models were varied between 350 mm and 8110 mm. Overall, a total of 160 numerical data on high-chromium stainless steel welded I-section columns were generated through parametric studies.

6.4 Assessment of international design standards

6.4.1 General

In this section, the obtained test and numerical results were employed to evaluate the applicability of the current design rules, as set out in the European code EN 1993-1-4 (CEN, 2015) and American design guide AISC 27 (AISC, 2013), to welded I-section columns made of the new grade EN 1.4420 high-chromium stainless steel. Note that all the partial safety factors adopted in the two considered design standards were taken as unity herein, resulting in unfactored design resistances $N_{u,pred}$ being

evaluated. Table 6.4 reports the mean test and numerical to predicted failure load ratio $N_u/N_{u,pred}$ and the corresponding COV for each considered design standard.

6.4.2 Eurocode EN 1993-1-4

The current Eurocode EN 1993-1-4 (CEN, 2015) adopts buckling curves, which were derived based on the Perry-Robertson buckling formulae (Ayrton & Perry, 1886; Robertson, 1925), for the design of welded, cold-formed and hot-rolled stainless steel column members failing by global instability (e.g., flexural, torsional and flexural-torsional buckling). With regard to stainless steel slender (Class 4) section columns susceptible to global buckling interacting with local buckling, the buckling curves are used in combination with the effective width method (for the treatment of slender sections failing by local buckling) to account for the local–global interactive buckling. For stainless steel non-slender (Class 1, 2 and 3) welded I-section column failing by flexural buckling, the EC3 design member resistance $N_{u,EC3}$ is calculated as the product of the cross-section yield load $A\sigma_{0.2}$ and a reduction factor χ , while the EC3 design compression resistance of a stainless steel slender welded I-section column susceptible to local–flexural interactive buckling $N_{u,EC3}$ is given as the product of the effective cross-section compression resistance $A_{eff,EC3}\sigma_{0.2}$ (for local buckling) and the reduction factor χ (for global buckling), as given by Equation (6.2).

$$N_{u,EC3} = \begin{cases} \chi A \sigma_{0.2} & \text{for Class 1, 2 and 3 cross-sections} \\ \chi A_{eff,EC3} \sigma_{0.2} & \text{for Class 4 cross-sections} \end{cases} \quad (6.2)$$

The effective cross-section area $A_{eff,EC3}$ is calculated as the sum of the effective area of each constituent plate element, with the formulae reported in detail in Section 3.4.2. The reduction factor χ can be calculated from the design buckling curves, as defined by Equation (6.3), where $\bar{\lambda}_{EC3}$ is the EC3 member non-dimensional slenderness about the buckling axis and can be calculated from Equation (6.4), while ϕ is a buckling parameter, as given by Equation (6.5), where α is the imperfection factor

and $\bar{\lambda}_0$ is the limiting slenderness, both of which are used to consider the effects of the membrane residual stresses and initial geometric imperfections on the compression resistances of stainless steel columns; for stainless steel welded I-section columns with flexural buckling about the minor principal axis, $\alpha=0.76$ and $\bar{\lambda}_0 = 0.20$.

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{EC3}^2}} \leq 1 \quad (6.3)$$

$$\bar{\lambda}_{EC3} = \begin{cases} \sqrt{\frac{A\sigma_{0.2}L_e^2}{\pi^2 EI}} & \text{for Class 1, 2 and 3 cross-sections} \\ \sqrt{\frac{A_{eff,EC3}\sigma_{0.2}L_e^2}{\pi^2 EI}} & \text{for Class 4 cross-sections} \end{cases} \quad (6.4)$$

$$\phi = 0.5 \left[1 + \alpha (\bar{\lambda}_{EC3} - \bar{\lambda}_0) + \bar{\lambda}_{EC3}^2 \right] \quad (6.5)$$

The applicability of the EC3 design rules for normal stainless steel welded I-section columns to their high-chromium counterparts was evaluating through comparing the predicted compression resistances with the test and numerical failure loads. A graphical evaluation was firstly performed, as presented in Figure 6.11, where the test and numerical failure loads, normalised by the cross-section yield loads $A\sigma_{0.2}$ (for non-slender welded I-sections) or the EC3 effective cross-section compression resistances $A_{eff,EC3}\sigma_{0.2}$ (for slender welded I-sections), are plotted against the EC3 member non-dimensional slendernesses $\bar{\lambda}_{EC3}$, together with the EC3 design buckling curve for normal stainless steel welded I-section columns failing by minor-axis flexural buckling. The results of the graphical evaluation indicated that the EC3 design buckling curve generally well captures all the test and numerical data points. Figure 6.12 depicts the ratios of the test and FE failure loads to the EC3 design compression resistances plotted against the EC3 member non-dimensional slendernesses $\bar{\lambda}_{EC3}$, while Table 6.4 reports the mean test and numerical to EC3 predicted failure load ratio $N_u/N_{u,EC3}=1.17$, with the corresponding COV of 0.05. On

the basis of the results of both the graphical and quantitative evaluations, it can thus be concluded that the current Eurocode EN 1993-1-4 (CEN, 2015) results in safe, accurate and consistent compression resistance predictions when applied to both the new high-chromium stainless steel non-slender welded I-section columns failing by flexural buckling and their slender welded I-section counterparts susceptible to local–flexural interactive buckling.

Figure 6.11: Comparison of normalised test and numerical failure loads with EC3 design buckling curve

Table 6.4: Comparisons of test and numerical results with compression resistance predictions from considered design standards

Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,EC3}$		$N_u/N_{u,AISC}$	
			Mean	COV	Mean	COV
Non-slender cross-section	10	70	1.16	0.05	1.20	0.12
Slender cross-section	10	90	1.17	0.05	1.23	0.11
Total	20	160	1.17	0.05	1.21	0.11

Figure 6.12: Comparison of EC3 and AISC design compression resistance predictions for high-chromium stainless steel welded I-section columns

6.4.3 American design guide AISC 27

The current American design guide AISC 27 (AISC, 2013) specifies that the load-carrying capacity of a compression member with uniform cross-section is calculated as the product of the member critical stress F_{cr} and the gross cross-section area A , as given by Equation (6.6), where the critical stress F_{cr} is dependent on the AISC member non-dimensional slenderness $\bar{\lambda}_{AISC}$ and can be calculated from Equations (6.7) and (6.8) for non-slender section columns and slender section columns, respectively, where $\bar{\lambda}_{AISC}$ is the AISC member non-dimensional slenderness, as given by Equation (6.9), and Q_a and Q_s are the net section reduction factors to take into account local buckling of slender web and flange of the I-section, respectively, as defined in Section 3.4.3.

$$N_{u,AISC} = F_{cr} A \quad (6.6)$$

$$F_{cr} = \begin{cases} 0.50 \bar{\lambda}_{AISC}^2 \sigma_{0.2} & \text{for } \bar{\lambda}_{AISC} \leq 1.2 \\ \frac{0.531}{\bar{\lambda}_{AISC}^2} \sigma_{0.2} & \text{for } \bar{\lambda}_{AISC} > 1.2 \end{cases} \quad (6.7)$$

$$F_{cr} = \begin{cases} 0.50 \bar{\lambda}_{AISC}^2 Q_s Q_a \sigma_{0.2} & \text{for } \bar{\lambda}_{AISC} \leq 1.2 \\ \frac{0.531}{\bar{\lambda}_{AISC}^2} Q_s Q_a \sigma_{0.2} & \text{for } \bar{\lambda}_{AISC} > 1.2 \end{cases} \quad (6.8)$$

$$\bar{\lambda}_{AISC} = \begin{cases} \sqrt{\frac{A \sigma_{0.2} L_e^2}{\pi^2 EI}} & \text{for non-slender sections} \\ \sqrt{\frac{Q_s Q_a A \sigma_{0.2} L_e^2}{\pi^2 EI}} & \text{for slender sections} \end{cases} \quad (6.9)$$

The applicability of the AISC design rules to the new high-chromium stainless steel welded I-section columns was evaluated, based on the test and numerical failure loads. Figure 6.13 depicts the normalised test and numerical failure loads plotted against the AISC member non-dimensional slendernesses $\bar{\lambda}_{AISC}$. The AISC design buckling curve, as defined by Equations (6.7) and (6.8) and also plotted in Figure 6.13, was generally shown to capture the test and numerical data points over the whole considered range of member non-dimensional slendernesses, although some disparity was found in the low slenderness range. The mean test and numerical to AISC predicted failure load ratio $N_u/N_{u,AISC}$ and the corresponding COV, as reported in Table 6.4, equal to 1.21 and 0.11, respectively, while comparisons between the AISC and EC3 design compression resistance predictions are shown in Figure 6.12. The American design guide AISC 27 (AISC, 2013) was shown to yield more conservative and scattered design compression resistance predictions than the European code EN 1993-1-4 (CEN, 2015), when used for the new high-chromium stainless steel welded I-section columns.

Figure 6.13: Comparison of normalised test and numerical failure loads with AISC design buckling curve

6.5 Reliability analysis

The reliability of the EC3 and AISC design rules when applied to the new high-chromium stainless steel welded I-section columns was evaluated in this section, according to the requirements and procedures given in EN 1990 (CEN, 2002) and AISC 27 (AISC, 2013), as detailed in Section 3.5. The key statistical parameters calculated according to EN 1990 (CEN, 2002) are reported in Table 6.5, where γ_{M1} is the partial safety factor for stainless steel member resistance, while Table 6.6 reports the key reliability analysis results for AISC 27 (AISC, 2013). The resulting (required) partial safety factors for the EC3 design rules, as given in Table 6.5, are less than the current used value of 1.1 in EN 1993-1-4 (CEN, 2015), while the resulting (required) resistance factor for the AISC design rules, as reported in Table 6.6, is larger than the target value of 0.9 in AISC 27 (AISC, 2013), therefore demonstrating the reliability of the EC3 and AISC design rules when applied to high-chromium stainless steel

welded I-section columns.

Table 6.5: 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}
EN 1993-1-4	180	3.145	1.173	0.044	0.090	0.87

Table 6.6: Reliability analysis results calculated according to AISC 27

Design method	No. of test and FE data	P_m	V_Q	V_R	β	ϕ_{AISC}
AISC 27	180	1.21	0.19	0.14	2.60	1.27

6.6 Concluding remarks

An in-depth testing and numerical modelling programme has been performed to investigate the structural behaviour and resistances of high-chromium stainless steel non-slender welded I-section columns (failing by minor-axis flexural buckling) and slender welded I-section columns (failing by local–flexural interactive buckling). Four high-chromium stainless steel welded I-sections were adopted in the testing programme, and for each cross-section, five column specimens with varying member lengths were prepared. The overall testing programme included initial global and local geometric imperfection measurements and pin-ended column tests on twenty high-chromium stainless steel welded I-section column specimens. The testing programme was followed by a numerical modelling programme, where FE models were firstly developed to simulate the experimental structural responses and then adopted to generate further numerical data over a wide range of cross-section dimensions and member effective lengths. The codified design rules for normal stainless steel welded I-section columns, as set out in EN 1993-1-4 (CEN, 2015) and AISC 27 (AISC, 2013), were discussed, and then evaluated for their applicability to the new high-chromium stainless steel welded I-section columns, based on the test and numerical data. The results of the evaluation generally revealed that (i) the EC3 design rules lead to safe, accurate and consistent compression resistance predictions

when used for the new high-chromium stainless steel welded I-section columns and (ii) the AISC critical stress method is also applicable to the new high-chromium stainless steel welded I-section columns, but with a slightly high degree of conservatism and scatter. Finally, statistical analyses were performed to confirm the reliability of the design rules given in EN 1993-1-4 (CEN, 2015) and AISC 27 (AISC, 2013).

CHAPTER 7

BUCKLING OF S690 HIGH STRENGTH STEEL WELDED I-SECTION COLUMNS

7.1 Introduction

This chapter presents a comprehensive experimental and numerical investigation of the flexural and local–flexural interactive buckling behaviour of S690 high strength steel welded I-section columns. A testing programme, adopting four different welded I-sections, was firstly conducted, and included initial global and local geometric imperfection measurements and twenty pin-ended column tests. Following the testing programme, a numerical modelling programme was performed, where column finite element models were developed and validated against the test results, and then employed to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions and member effective lengths. The obtained test and numerical data, together with the test data collected from previous relevant literature (Rasmussen & Hancock, 1995; Li et al., 2016; Ma et al., 2018b), were employed to evaluate the accuracy of the design rules, as given in the current European code (CEN, 2007), American specification (AISC, 2016) and Australian standard (AS, 2016), for S690 high strength steel welded I-section columns failing by flexural buckling and local–flexural interactive buckling. The findings of this research work have been reported in Sun et al. (2020c, 2021a).

7.2 Experimental study

7.2.1 General

An experimental study was firstly conducted to investigate the structural behaviour and load-carrying capacities of S690 high strength steel non-slender welded I-section columns failing by minor-axis flexural buckling and slender welded I-section columns failing by local–flexural interactive buckling. The experimental study employs four different I-sections – I-50×50×5, I-80×60×5, I-100×100×5 and I-140×70×5, which were fabricated from 5 mm thick S700MC high strength steel hot-rolled plates by robotic GMAW. Specifically, I-50×50×5 and I-80×60×5 are respectively categorised as Class 1 and Class 2 (i.e. non-slender sections), according to the slenderness limits given in EN 1993-1-12 (CEN, 2007), and also classified as non-slender sections, in accordance with both the limiting width-to-thickness ratios given in ANSI/AISC 360-16 (AISC, 2016) and the yield slenderness limits specified in AS 4100 (AS, 2016); both of I-100×100×5 and I-140×70×5 are defined as Class 4 cross-sections, based on the slenderness limits given in EN 1993-1-12 (CEN, 2007), and also categorised as slender cross-sections, according to the limiting width-to-thickness ratios set out in ANSI/AISC 360-16 (AISC, 2016) and the yield slenderness limits given AS 4100 (AS, 2016). It is worth noting that the material properties and membrane residual stresses of the studied S690 high strength steel welded I-sections have been experimentally derived and fully reported in Chapter 4. The overall testing programme included twenty pin-ended column tests and supplementary initial geometric imperfection measurements.

7.2.2 Pin-ended column tests

Compression tests of pin-ended S690 high strength steel non-slender and slender welded I-section columns were conducted to investigate their flexural buckling and

local–flexural interactive buckling behaviour. For each of the four employed welded I-sections, five column specimens with varying member lengths were adopted in the testing programme, resulting in a total of twenty pin-ended column tests. The key geometric properties and initial global and local geometric imperfections of the S690 high strength steel welded I-section column specimens were measured, as reported in Table 7.1; note that the test setup and procedures of the initial geometric imperfection measurements were in line with those detailed in Section 6.2.2. All the column specimens were tested using an INSTRON 5000 kN servo-hydraulic testing machine at a rate of 0.4 mm/min. The employed column test setup and procedures were the same as those detailed in Section 6.2.3. The load–mid-height lateral deflection curves for the tested S690 high strength steel welded I-section column specimens with non-slender and slender cross-sections are shown in Figure 7.1 and Figure 7.2, respectively, while Table 7.2 reports the key obtained test results, including the final overall global geometric imperfection magnitude (ω_g+e_0) , together with the normalised value $(\omega_g+e_0)/L_e$, the failure load N_u and the mid-height lateral deflection at the failure load Δ_u . Regarding the failure modes of the tested S690 high strength steel column specimens, all the non-slender welded I-section column specimens failed by member flexural buckling about the minor principal axis, with typical failed specimens I-80×60×5-B1, I-80×60×5-B3 and I-80×60×5-B5 with relatively small, intermediate and large non-dimensional member slendernesses shown in Figure 7.3, while all the slender welded I-section column specimens displayed visible member flexural buckling about the minor principal axis coupled with local buckling of compression flanges, i.e. local–minor-axis flexural interactive buckling, with typical failed specimens I-100×100×5-C2, I-100×100×5-C3 and I-100×100×5-C4 shown in Figure 7.4.

Table 7.1: Measured geometric dimensions and initial geometric imperfection magnitudes of S690 high strength steel welded I-section column specimens

Cross-section	Specimen ID	L (mm)	L_e (mm)	h (mm)	b_f (mm)	t (mm)	ω_0 (mm)	ω_g (mm)
I-50×50×5	A1	239	349	49.38	49.48	4.96	0.05	0.11
	A2	340	450	49.69	49.44	5.00	0.05	0.20
	A3	450	560	47.33	49.46	4.95	0.05	0.20
	A4	578	688	49.36	49.55	4.92	0.05	0.29
	A5	696	806	49.54	49.45	4.96	0.05	0.31
I-80×60×5	B1	299	409	80.09	59.39	4.97	0.10	0.12
	B2	469	579	79.54	58.96	4.99	0.10	0.17
	B3	596	706	79.43	59.12	4.93	0.10	0.20
	B4	650	760	79.99	59.27	4.97	0.10	0.25
	B5	720	830	78.06	58.99	4.95	0.10	0.37
I-100×100×5	C1	455	565.1	99.49	99.14	4.98	0.11	0.19
	C2	598	708.0	99.26	99.04	5.00	0.11	0.22
	C3	692	801.8	99.47	99.12	4.98	0.11	0.26
	C4	820	930.0	99.44	99.00	4.98	0.11	0.36
	C5	949	1059.0	99.16	98.98	4.99	0.11	0.40
I-140×70×5	D1	446	555.7	139.50	69.16	5.03	0.08	0.11
	D2	496	606.0	139.66	69.20	4.95	0.08	0.18
	D3	598	708.0	139.14	69.26	4.97	0.08	0.28
	D4	670	780.2	139.72	69.04	4.98	0.08	0.26
	D5	778	888.2	139.93	69.26	4.93	0.08	0.37

Table 7.2: Key test results for S690 high strength steel welded I-section column specimens

Cross-section	Specimen ID	ω_g+e_0 (mm)	$(\omega_g+e_0)/L_e$	N_u (kN)	Δ_u (mm)
I-50×50×5	A1	0.30	1/1164	506.9	1.6
	A2	0.38	1/1197	446.8	2.3
	A3	0.41	1/1366	410.2	1.7
	A4	0.43	1/1600	331.3	2.1
	A5	0.50	1/1612	271.7	2.6
I-80×60×5	B1	0.38	1/1076	622.9	2.4
	B2	0.37	1/1565	554.2	3.1
	B3	0.52	1/1358	504.7	3.2
	B4	0.58	1/1310	461.7	3.2
	B5	0.63	1/1317	418.8	2.6
I-100×100×5	C1	0.29	1/1948	1021.6	1.5
	C2	0.32	1/2212	975.4	1.8
	C3	0.41	1/1956	948.4	2.2
	C4	0.54	1/1722	905.0	2.4
	C5	0.52	1/2037	818.1	2.8
I-140×70×5	D1	0.32	1/1737	860.9	1.4
	D2	0.36	1/1683	777.1	1.6
	D3	0.52	1/1362	686.5	1.6
	D4	0.51	1/1530	628.4	3.4
	D5	0.67	1/1326	582.5	4.2

(a) I-50x50x5 column specimens

(b) I-80x60x5 column specimens

Figure 7.1: Load–mid-height lateral deflection curves for S690 high strength steel non-slender welded I-section column specimens

(a) I-100x100x5 column specimens

(b) I-140x70x5 column specimens

Figure 7.2: Load–mid-height lateral deflection curves for S690 high strength steel slender welded I-section column specimens

Figure 7.3: Minor-axis flexural buckling failure modes of S690 high strength steel non-slender welded I-section column specimens (from left to right: I-80×60×5-B1, I-80×60×5-B3 and I-80×60×5-B5)

Figure 7.4: Local–flexural interactive buckling failure modes of typical S690 high strength steel slender welded I-section column specimens (from left to right: I-100×100×5-C2, 100×100×5-C3 and 100×100×5-C4)

7.3 Numerical study

7.3.1 General

In conjunction with the testing programme, a numerical modelling programme was performed, using the nonlinear finite element software ABAQUS (2014). FE models were firstly developed and validated against the test results, and then employed to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions and member effective lengths.

7.3.2 Development and validation of finite element models

Each S690 high strength steel welded I-section column FE model was developed based on the measured cross-section geometric dimensions and member effective length, with the same modelling assumptions, procedures and techniques as those described in Section 6.3.2 employed herein. Regarding the incorporation of initial global and local geometric imperfections, three global imperfection magnitudes, including the measured overall global geometric imperfection magnitude ($\omega_g + e_0$) and two fractions of the member effective length ($L_e/1000$ and $L_e/1500$), were used to scale the corresponding initial global imperfection distribution pattern of each column FE model, while three local imperfection magnitudes, including the measured magnitude ω_0 and two fractions of the plate thickness ($t/10$ and $t/100$), were used to scale the corresponding initial local imperfection distribution pattern. Five sets of initial global and local geometric imperfection magnitude combinations, including ($\omega_g + e_0$) and ω_0 , $L_e/1000$ and $t/10$, $L_e/1000$ and $t/100$, $L_e/1500$ and $t/10$, and $L_e/1500$ and $t/100$, were considered to evaluate the sensitivity of the developed S690 high strength steel welded I-section column FE models to geometric imperfection magnitudes and seek the most appropriate imperfection magnitude combination to be adopted in the parametric studies.

Upon development of the S690 high strength steel welded I-section column FE models, the modified Riks method (ABAQUS, 2014), taking into account both the material and geometric nonlinearities, was performed to derive the numerical failure loads, load–mid-height lateral deflection curves and numerical failure modes. The numerically derived results were compared with their experimentally obtained results to evaluate the accuracy of the developed column FE models. Table 7.3 reports the numerical to test failure load ratios for the column specimens, with the results generally indicating that (i) the test failure loads are well predicted for all the five adopted initial global and local geometric imperfection magnitude combinations, and (ii) the best agreement between the test and numerical failure loads is obtained when the initial global and local imperfection magnitudes are taken as $L_e/1000$ and $t/100$, respectively. Comparisons of the test load–mid-height lateral deflection curves with their numerical counterparts for a typical non-slender welded I-section column specimen I-50×50×5-A4 and a typical slender welded I-section column specimen I-100×100×5-C4 are depicted in Figure 7.5 and Figure 7.6, respectively, where the test load–mid-height lateral deflection curves are well captured by their numerical counterparts. The test and numerical minor-axis flexural buckling failure modes for the non-slender welded I-section column specimen I-50×50×5-A4 are compared in Figure 7.7, while Figure 7.8 displays the test and numerical local–global (minor-axis flexural) interactive buckling failure modes for the slender welded I-section column specimen I-100×100×5-C4, showing excellent agreement. Overall, it can be concluded that the developed column FE models are capable of simulating the test responses of the S690 high strength steel welded I-section column specimens, and therefore deemed to be validated.

Table 7.3: Comparison of test and FE failure loads varying global and local geometric imperfection magnitude combinations

Cross-section	Specimen ID	FE N_u / Test N_u				
		(ω_g+e_0) $+\omega_0$	$L_e/1000$ $+t/10$	$L_e/1000$ $+t/100$	$L_e/1500$ $+t/10$	$L_e/1500$ $+t/100$
I-50×50×5	A1	1.058	1.031	1.036	1.065	1.070
	A2	1.050	1.010	1.015	1.064	1.068
	A3	0.980	0.938	0.944	1.003	1.007
	A4	0.987	0.965	0.969	0.976	0.979
	A5	1.010	0.982	0.985	0.995	0.998
I-80×60×5	B1	1.044	1.024	1.030	1.064	1.069
	B2	1.047	1.011	1.015	1.038	1.043
	B3	0.958	0.935	0.939	0.969	0.973
	B4	1.035	1.005	1.008	1.047	1.050
	B5	0.980	0.971	0.974	0.986	0.988
I-100×100×5	C1	1.054	1.037	1.042	1.046	1.050
	C2	1.045	1.023	1.027	1.034	1.037
	C3	0.963	0.948	0.952	0.957	0.960
	C4	1.038	1.017	1.019	1.033	1.034
	C5	0.981	0.973	0.976	0.977	0.979
I-140×70×5	D1	0.998	0.984	0.988	0.993	0.996
	D2	1.030	1.008	1.011	1.024	1.026
	D3	0.974	0.956	0.960	0.975	0.978
	D4	1.008	0.991	0.994	1.005	1.007
	D5	1.027	1.012	1.015	1.030	1.031
	Mean	1.013	0.991	0.995	1.014	1.017
	COV	0.033	0.032	0.032	0.034	0.034

Figure 7.5: Test and numerical load–mid-height lateral deflection curves for S690 high strength steel non-slender welded I-section column specimen I-50×50×5-A4

Figure 7.6: Test and numerical load–mid-height lateral deflection curves for S690 high strength steel non-slender welded I-section column specimen I-100×100×5-C4

Figure 7.7: Test and numerical minor-axis flexural buckling failure modes for column specimen I-50×50×5-A4

Figure 7.8: Test and numerical local–flexural interactive buckling failure modes for column specimen I-100×100×5-C4

7.3.3 Parametric studies

Having been successfully validated, the developed column FE models were adopted to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions and member effective lengths. Regarding the cross-section dimensions of the numerically modelled welded I-sections, the outer section heights were fixed at 200 mm, and the flange widths were varied between 50 mm and 200 mm, with the plate thicknesses set to be equal and ranging from 3.5 mm to 10 mm, leading to a wide range of cross-section aspect ratios and dimensions being considered. Note that all the numerically modelled I-sections cover all four classes (i.e. Class 1, Class 2, Class 3 and Class 4) of cross-sections, according to the slenderness limits set out in EN 1993-1-2 (CEN, 2007). The effective lengths of the column FE models fell between 260 mm and 4010 mm, resulting in a broad range of member slendernesses being accounted for. Given that the initial global and local geometric imperfection magnitude combination – $L_e/1000$ and $t/100$ leads to the best agreement between the test and numerical failure loads in the validation study, these two magnitudes were used for the column FE models in the parametric studies. In total, 80 numerical data on S690 high strength steel non-slender welded I-section columns and 70 numerical data on S690 high strength steel slender welded I-section columns were generated through parametric studies.

7.4 Evaluation of current design standards and development of new design approaches

7.4.1 General

In this section, the current design rules, as set out in the European code EN 1993-1-2 (CEN, 2007), Australian standard AS 4100 (AS, 2016) and American specification

ANSI/AISC 360-16 (AISC, 2016), for S690 high strength steel welded I-section columns were fully described and discussed. The accuracy of each design standard was then evaluated by comparing the test and numerical failure loads N_u with the unfactored design compression resistances $N_{u,pred}$. Tables 7.4–7.6 report the mean test and numerical to predicted failure load ratio for the three considered design standards (CEN, 2007; AS, 2016; AISC, 2016), together with the corresponding COVs. The shortcomings of the three considered design standards were then highlighted, followed by the development of new design approaches.

7.4.2 European code EN 1993-1-12

The current Eurocode EN 1993-1-12 (CEN, 2007) employs buckling curves, as developed in accordance with the Perry-Robertson buckling formulae (Ayrton & Perry, 1886; Robertson, 1925), for the design of high strength steel column members susceptible to global instability (e.g., flexural, torsional and flexural-torsional buckling); regarding high strength steel columns failing by local–global interactive buckling, EN 1993-1-12 (CEN, 2007) combines the use of effective width method and buckling curves to account for the interaction of local and global buckling in the design. For high strength steel non-slender (Class 1, 2 and 3) welded I-section column failing by flexural buckling, the EC3 design member resistance $N_{u,EC3}$ is given as the product of the cross-section yield load Af_y and a reduction factor χ , while the EC3 design compression resistance of a high strength steel slender welded I-section column susceptible to local–flexural interactive buckling $N_{u,EC3}$ is calculated as the product of the effective cross-section compression resistance $A_{eff,EC3}f_y$ (for local buckling) and the reduction factor χ (for global buckling), as given by Equation (7.1).

$$N_{u,EC3} = \begin{cases} \chi Af_y & \text{for Class 1, 2 and 3 cross-sections} \\ \chi A_{eff,EC3} f_y & \text{for Class 4 cross-sections} \end{cases} \quad (7.1)$$

The effective cross-section area $A_{eff,EC3}$ can be determined by means of the EC3

effective width method, which takes into account loss of effectiveness of a slender I-section failing by local buckling through reductions in flat widths of slender constituent plate elements, as detailed in Section 4.4.2; the reduction factor χ considers the influence of global buckling on the compression resistances of columns and can be determined through the EC3 design buckling curve, as defined by Equation (7.2), where $\bar{\lambda}_{EC3}$ is the EC3 member non-dimensional slenderness about the considered axis of buckling, as calculated by Equation (7.3), and ϕ is the buckling parameter, as determined from Equation (7.4), where α is the imperfection factor to consider the effects of residual stresses and initial geometric imperfections on the compression resistances of columns; note that the value of α is dependent on the adopted design buckling curve, and for the studied S690 high strength steel welded I-section columns failing by minor-axis flexural buckling, design buckling curve ‘c’ (CEN, 2007) is employed, with $\alpha=0.49$.

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{EC3}^2}} \leq 1 \quad (7.2)$$

$$\bar{\lambda}_{EC3} = \begin{cases} \sqrt{\frac{Af_y L_e^2}{\pi^2 EI}} & \text{for Class 1, 2 and 3 cross-sections} \\ \sqrt{\frac{A_{eff,EC3} f_y L_e^2}{\pi^2 EI}} & \text{for Class 4 cross-sections} \end{cases} \quad (7.3)$$

$$\phi = 0.5 \left[1 + \alpha (\bar{\lambda}_{EC3} - 0.2) + \bar{\lambda}_{EC3}^2 \right] \quad (7.4)$$

The accuracy of the EC3 design rules for S690 high strength steel welded I-section columns was evaluated, based on the obtained test and numerical data and the test results collected from previous relevant studies (Rasmussen & Hancock, 1995; Li et al., 2016; Ma et al., 2018b). Figure 7.9 depicts the normalised test and numerical failure loads (by the EC3 design cross-section compression resistances Af_y or $A_{eff,EC3}f_y$) plotted against the member non-dimensional slendernesses $\bar{\lambda}$, together with the EC3

design buckling curve ‘c’; it was generally found that the EC3 design buckling curve ‘c’ lies well below the test and numerical data points for S690 high strength steel welded I-section columns, revealing conservatism. The ratios of the test and numerical failure loads to the EC3 design compression resistances are plotted against the member non-dimensional slendernesses $\bar{\lambda}$ in Figure 7.10, while the mean test and numerical to EC3 predicted resistance ratios and the corresponding COVs are listed in Table 7.4. Both the graphical and quantitative evaluation results indicated that the current European code EN 1993-1-12 (CEN, 2007) leads to safe but rather conservative resistance predictions for both S690 high strength steel non-slender and slender welded I-section columns.

Figure 7.9: Comparisons of test and numerical failure loads with EC3 design flexural buckling curve and new design proposal

Figure 7.10: Comparisons of test and numerical results with compression resistance predictions from EN 1993-1-12 and proposed design approach

Table 7.4: Comparisons of test and numerical results with EN 1993-1-12 compression resistance predictions

Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,EC3}$	
			Mean	COV
Non-slender sections	27	80	1.24	0.09
Slender sections	10	70	1.22	0.06
Total	37	150	1.23	0.07

7.4.3 Australian standard AS 4100

AS 4100 (AS, 2016) adopts the same design concepts for welded I-section columns as those employed in EN 1993-1-12 (CEN, 2007), although the design formulations are different. Specifically, AS 4100 (AS, 2016) uses the design concept of buckling curve for the design of non-slender welded I-section columns failing by flexural buckling, while combines the use of effective width method and buckling curve to account for the interaction of local and flexural buckling in the design of slender

welded I-section columns. The AS design compression resistance $N_{u,AS}$ of a welded I-section column is determined as the product of the AS design cross-section compression resistance (Af_y or $A_{eff,AS}f_y$) and flexural buckling reduction factor α_c , as given by Equation (7.5),

$$N_{u,AS} = \begin{cases} \alpha_c Af_y & \text{for non-slender sections} \\ \alpha_c A_{eff,AS} f_y & \text{for slender sections} \end{cases} \quad (7.5)$$

where the effective cross-section area of the slender I-section $A_{eff,AS}$ is calculated according to the AS effective width method, as detailed in Section 4.4.2, and the flexural buckling reduction factor α_c , as defined by the AS design buckling curve, is given by Equation (7.6), where λ_n is the modified member slenderness, as calculated from Equation (7.7), where $\bar{\lambda}_{AS}$ is the AS member non-dimensional slenderness, as defined by Equation (7.8), α_a is the compression member factor, as given by Equation (7.9), α_b is the member section constant and equal to 0.5 for I-sections fabricated from hot-rolled sheets, and ξ is a buckling parameter and can be determined from Equation (7.10), where η is the compression member imperfection factor, as given by Equation (7.11).

$$\alpha_c = \xi \left[1 - \sqrt{1 - \left[\frac{90}{\xi(\lambda_n + \alpha_a \alpha_b)} \right]^2} \right] \leq 1 \quad (7.6)$$

$$\lambda_n = \sqrt{\frac{\pi^2 E}{250}} \bar{\lambda}_{AS} \quad (7.7)$$

$$\bar{\lambda}_{AS} = \begin{cases} \sqrt{\frac{Af_y L_e^2}{\pi^2 EI}} & \text{for non-slender sections} \\ \sqrt{\frac{A_{eff,AS} f_y L_e^2}{\pi^2 EI}} & \text{for slender sections} \end{cases} \quad (7.8)$$

$$\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\lambda_n + 2050} \quad (7.9)$$

$$\xi = \frac{\left(\frac{\lambda_n + \alpha_a \alpha_b}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda_n + \alpha_a \alpha_b}{90}\right)^2} \quad (7.10)$$

$$\eta = 0.00326(\lambda_n + \alpha_a \alpha_b - 13.5) \geq 0 \quad (7.11)$$

The AS design compression resistances for S690 high strength steel welded I-section columns were compared with the obtained test and numerical failure loads and the test failure loads collected from previous studies (Rasmussen & Hancock, 1995; Li et al., 2016; Ma et al., 2018b). Figure 7.11 depicts the normalised test and numerical failure loads (by the AS design cross-section compression resistances Af_y or $A_{eff,AS}f_y$) plotted against the AS member non-dimensional slendernesses $\bar{\lambda}_{AS}$, together with the AS design buckling curve, as defined by Equations (7.6)–(7.11). The AS design buckling curve was shown to lie considerably below the test and numerical data points, indicating a high level of conservatism. Moreover, as shown in Table 7.5 and Figure 7.12, the AS design compression resistances are even less accurate than the EC3 design compression resistance predictions, though with a similar degree of scatter.

Table 7.5: Comparisons of test and numerical results with AS 4100 compression resistance predictions

Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,AS}$	
			Mean	COV
Non-slender sections	27	80	1.26	0.09
Slender sections	10	70	1.29	0.06
Total	37	150	1.27	0.07

Figure 7.11: Comparisons of test and numerical failure loads with AS design flexural buckling curve

Figure 7.12: Comparisons of test and numerical results with compression resistance predictions from AS 4100 and proposed design approach

7.4.4 American specification ANSI/AISC 360-16 (AISC)

The AISC design compression resistance ($N_{u,AISC}$) of a welded I-section column is calculated from Equation (7.12), where $A_{eff,AISC}$ is the effective cross-section area, which can be determined according to the AISC effective width method, as described in Section 4.4.2, and F_{cr} is the critical stress, as calculated from the design buckling curve defined by Equation (7.13), where $\bar{\lambda}_{AISC}$ is the AISC member non-dimensional slenderness, as given by Equation (7.14).

$$N_{u,AISC} = \begin{cases} F_{cr} A & \text{for non-slender sections} \\ F_{cr} A_{eff,AISC} & \text{for slender sections} \end{cases} \quad (7.12)$$

$$F_{cr} = \begin{cases} 0.658^{\bar{\lambda}_{AISC}^2} f_y & \text{for } \bar{\lambda}_{AISC} \leq 1.5 \\ \frac{0.877}{\bar{\lambda}_{AISC}^2} f_y & \text{for } \bar{\lambda}_{AISC} > 1.5 \end{cases} \quad (7.13)$$

$$\bar{\lambda}_{AISC} = \sqrt{\frac{Af_y L_e^2}{\pi^2 EI}} \quad (7.14)$$

The accuracy of the AISC design rules for S690 high strength steel non-slender welded I-section columns failing by flexural buckling and their slender counterparts susceptible to local–flexural interactive buckling was assessed through comparisons against the test and numerical failure loads and collected from previous experiments (Rasmussen & Hancock, 1995; Li et al., 2016; Ma et al., 2018b). Figure 7.13 and Table 7.6 respectively present the graphical and quantitative assessment results, revealing that the current ANSI/AISC 360-16 (AISC, 2016) yields accurate design compression resistances on average, but results in some unsafe resistance predictions for S690 high strength steel welded I-section columns. This is also evident in Figure 7.14, where the normalised test and numerical failure loads, are compared against the AISC design buckling curve defined by Equation (7.13).

Figure 7.13: Comparisons of test and numerical results with compression resistance predictions from ANSI/AISC 360-16 and proposed design approach

Figure 7.14: Comparisons of test and numerical failure loads with ANSI/AISC 360-16 design flexural buckling curve

Table 7.6: Comparisons of test and numerical results with ANSI/AISC 360-16 compression resistance predictions

Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,AISC}$	
			Mean	COV
Non-slender sections	27	80	1.07	0.06
Slender sections	10	70	1.04	0.06
Total	37	150	1.05	0.06

7.4.5 New design approaches

Summing up of the graphical and quantitative evaluation results, it was concluded that both EN 1993-1-12 (CEN, 2007) and AS 4100 (AS, 2016) generally result in conservative resistance predictions for both S690 high strength steel non-slender welded I-section columns failing by minor-axis flexural buckling and S690 high strength steel slender welded I-section columns failing by local–flexural interactive buckling, while ANSI/AISC 360-16 (AISC, 2016) leads to overall accurate design compression resistances, but with some unsafe resistance predictions for S690 high strength steel welded I-section columns. This thus prompted the development of safe and accurate design approaches for S690 high strength steel welded I-section columns. Regarding S690 high strength steel welded I-section columns failing by minor-axis flexural buckling, an improved design approach was proposed herein through using the original EC3 design buckling curve, as defined by Equations (7.2)–(7.4), but with a reduced imperfection factor $\alpha = 0.21$, i.e. the adoption of the EC3 design buckling curve ‘a’ (CEN, 2007). Given that the EC3 effective width method was found to be generally accurate in previous studies of S690 high strength steel slender welded I-section stub columns failing by local buckling, as highlighted in Chapter 4, a new design approach was thus proposed herein for S690 high strength steel slender welded I-section columns susceptible to local–flexural interactive buckling by adopting the EC3 effective width method combined with the new design flexural buckling curve (i.e. EC3 buckling curve ‘a’).

The improved design buckling curve for S690 high strength steel welded I-section columns susceptible to minor-axis flexural buckling and local–minor-axis flexural interactive buckling is shown in Figure 7.9, exhibiting a good representation of the test and numerical data point over the whole member non-dimensional slenderness range. The new design approaches were shown to result in a substantial improvement in the design accuracy over EN 1993-1-12 (CEN, 2007) and AS 4100 (AS, 2016) – see Figure 7.10 and Figure 7.12, while in comparison with ANSI/AISC 360-16 (AISC, 2016), the new design approaches yield even more accurate resistance predictions for those relatively short S690 high strength steel welded I-section columns (with member non-dimensional slendernesses less than around 1.0) and safe-sided resistance predictions for their intermediate and long counterparts (with member non-dimensional slendernesses greater than around 1.0), as shown in Figure 7.13. Quantitative evaluation of the accuracy of the proposed design approaches was conducted through comparing the test and numerical failure loads N_u against the predicted flexural buckling resistances $N_{u,pr}$. The mean $N_u/N_{u,pr}$ ratios and the corresponding COVs are reported in Table 7.7, indicating that the proposed design approaches yield more accurate and consistent compression resistance predictions for S690 high strength steel welded I-section columns than the three international design standards (CEN, 2007; AS, 2016; AISC, 2016), and the predicted compression resistances also lie on the safe side.

Table 7.7: Comparisons of test and numerical results with compression resistance predictions from new design approaches

Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,pr}$	
			Mean	COV
Non-slender sections	27	80	1.05	0.06
Slender sections	10	70	1.05	0.03
Total	37	150	1.05	0.05

7.5 Reliability analysis

The reliability of the new design approaches was assessed in accordance with the requirements and procedures specified in EN 1990 (CEN, 2002). In the reliability analysis, the COV of geometric dimensions of high strength steel members was taken as 0.02 (Bartlette et al., 2003; Wang & Gardner, 2017), and the material over-strength ratio was taken as 1.135, with the COV of 0.055 (Wang & Gardner, 2017). The key obtained statistical parameters are reported in Table 7.8. The (required) partial safety factors for the new design approaches for S690 high strength steel non-slender and slender welded I-section columns are respectively equal to 1.04 and 1.03 (see Table 7.8), which is slightly larger than the currently adopted value of 1.00 in EN 1993-1-12 (CEN 2007). Previous studies (Charbrolin, 2002; Wang & Gardner, 2017) suggested that a higher target value of $\gamma_{M1}=1.10$ be adopted for high strength steel structures, which is supported by the results of the present reliability analysis.

Table 7.8: Summary of reliability analysis results for new design approaches

Cross-section category	Number of data	$k_{d,n}$	b	V_δ	V_r	γ_{M1}
Non-slender I-section	107	3.18	1.089	0.053	0.079	1.04
Slender I-section	80	3.21	1.049	0.028	0.065	1.03

7.6 Concluding remarks

A thorough experimental and numerical modelling programme has been performed to investigate the structural behaviour and resistances of S690 high strength steel non-slender welded I-section columns failing by minor-axis flexural buckling and their slender counterparts failing by local–flexural interactive buckling. The experimental programme was carried out on two non-slender welded I-sections and two slender welded I-sections, and included initial global and local geometric imperfection measurements and twenty pin-ended column tests. The obtained test results were adopted in a parallel numerical modelling programme for validation of the developed

column FE models, which were then used to conduct parametric studies to generate further numerical data on S690 high strength steel welded I-section columns susceptible to minor-axis flexural buckling and local–flexural interactive buckling. The obtained test and numerical results, together with the test data collected from the literature (Rasmussen & Hancock, 1995; Li et al., 2016; Ma et al., 2018b), were adopted to evaluate the accuracy of the relevant column buckling design rules, as given in the current European code EN 1993-1-12 (CEN, 2007), Australian standards AS 4100 (AS, 2016) and American specification ANSI/AISC 360-16 (AISC, 2016). The evaluation results generally revealed that both EN 1993-1-12 (CEN, 2007) and AS 4100 (AS, 2016) result in conservative resistance predictions for S690 high strength steel welded I-section columns, while ANSI/AISC 360-16 (AISC, 2016) leads to an overall high level of design accuracy, but with some over-predicted compression resistances. Finally, an improved design approach was developed through modifications to the EC3 design buckling curve for S690 high strength steel non-slender welded I-section columns failing by minor-axis flexural buckling, and shown to yield safe, accurate and consistent resistance predictions, while a new design approach, which combines the use of the improved design curve and the EC3 effective width method, was proposed for S690 high strength steel slender welded I-section columns susceptible to local–minor-axis flexural interactive buckling, and found to result in a high level of design accuracy and consistency.

CHAPTER 8

BUCKLING OF HIGH-CHROMIUM STAINLESS STEEL WELDED I-SECTION BEAM-COLUMNS

8.1 Introduction

This paper reports experimental and numerical investigations of the global instability of the new high-chromium stainless steel welded I-section beam-columns. A testing programme was firstly conducted, comprising initial geometric imperfections measurements and minor-axis eccentric compression tests on ten high-chromium stainless steel welded I-section beam-column specimens. The key test results, together with the test setups and procedures, were fully reported. Following the testing programme, a numerical modelling programme was carried out, where finite element models were developed and validated against the test results, and then used to perform parametric studies to generate additional numerical data over an extensive range of cross-section sizes, member lengths and loading combinations. On the basis of the obtained test and numerical results, the applicability of the design interaction curves for normal stainless steel welded I-section beam-columns, set out in the current Eurocode EN 1993-1-4 (CEN, 2015) and American design guide AISC 27 (AISC, 2013), to the new high-chromium stainless steel welded I-section beam-columns were evaluated. Following this, an improved design approach was developed. The findings of this research have been reported in Sun et al. (2021b).

8.2 Testing

8.2.1 General

Given that there have been no experimental results on the new high-chromium stainless steel welded I-section beam-columns, a testing programme was firstly carried out to establish a test data bank. The testing programme adopted two welded I-section sizes: I-150×100×6 and I-150×120×6, which are categorised as Class 1 and Class 3 cross-sections, respectively, based on the slenderness limits given in EN 1993-1-4 (CEN, 2015), and are also respectively classified as compact and non-compact cross-sections, in accordance with the limiting width-to-thickness ratios set out in AISC 27 (AISC, 2013). Ten beam-column specimens were prepared, with five for each adopted I-section size; all of them were fabricated by robotic GMAW from 6 mm thick high-chromium stainless steel plates. Note that the material properties and membrane residual stresses of the studied high-chromium stainless steel welded I-sections have been experimentally measured and presented in Chapter 3. The overall testing programme included initial geometric imperfection measurements and ten beam-column tests about the minor axis.

8.2.2 Beam-column tests

Minor-axis eccentric compression tests were conducted to investigate the global instability and load-carrying capacities of the new high-chromium stainless steel welded I-section beam-columns under combined compression and minor-axis bending moment. For each adopted I-section size, five geometrically identical beam-column specimens were tested with different initial loading eccentricities varied from 5 mm to 60 mm, leading to a range of loading combinations being considered. For each high-chromium stainless steel welded I-section beam-column specimen, its key geometric dimensions and initial global and local geometric imperfections were

measured before two 15 mm thick end plates were welded to its two end sections, as presented in Table 8.1. Note that the initial geometric imperfection measurements adopted the same test setup and procedures as those described in Section 6.2.2. An INSTRON 5000 kN testing machine was used to test all the beam-column specimens at a constant loading rate of 0.25 mm/min. The testing machine was equipped with knife-edge devices, which offered pin-ended boundary conditions to the beam-column specimen ends. As shown in Figure 8.1, each knife-edge device consisted of a pit plate with a semi-circular groove and a wedge plate with a knife-edge wedge. Each beam-column specimen welded with end plates was placed between the top and bottom wedge plates before testing, and the relative position between the specimen and the wedge plates was carefully adjusted by means of a laser and two spirit levels to ensure that the cross-section minor axis of the specimen was parallel to the knife edges, with the distance equal to the pre-specified initial loading eccentricity; moreover, the specimen was intersected with the wedge plates at right angles in the longitudinal direction to achieve good member alignment. Upon achievement of position adjustment and member alignment, the specimen was bolted to the wedge plates and placed between the pit plates. Note that the distance between the specimen end and the rotation centre of the knife-edge device was equal to 70 mm – see Figure 8.1; the member effective lengths of the beam-column specimens are thus calculated as $L_e = L + 140$ mm, as given in Table 8.1, and their member non-dimensional slendernesses $\bar{\lambda}$ about the minor axis are determined based on Equation (8.1) and presented in Table 8.1, where $N_y = A\sigma_{0.2}$ is the cross-section yield load, and $N_{cr} = \pi^2 EI_z / L_e^2$ is the Euler buckling load about the minor axis, in which I_z is the second moment of area about the minor axis.

$$\bar{\lambda} = \sqrt{\frac{N_y}{N_{cr}}} = \sqrt{\frac{A\sigma_{0.2}}{\frac{\pi^2 EI_z}{L_e^2}}} \quad (8.1)$$

Table 8.1: Measured geometric dimensions and initial geometric imperfection magnitudes of high-chromium stainless steel welded I-section beam-column specimens

Cross-section	Specimen ID	L (mm)	L_e (mm)	$\bar{\lambda}$	h (mm)	b_f (mm)	t (mm)	ω_0 (mm)	ω_g (mm)
I-150×100×6	A1	949	1089	0.66	149.74	99.60	5.98	0.04	0.20
	A2	950	1090	0.66	148.95	99.71	5.88	0.08	0.23
	A3	950	1090	0.66	149.56	99.14	5.91	0.06	0.19
	A4	950	1090	0.66	149.58	99.87	5.96	0.06	0.25
	A5	949	1089	0.66	149.52	99.20	5.92	0.04	0.20
I-150×120×6	B1	1200	1340	0.67	149.50	119.81	5.97	0.07	0.28
	B2	1198	1338	0.66	149.22	119.34	5.91	0.05	0.32
	B3	1199	1339	0.67	149.67	119.55	5.92	0.05	0.28
	B4	1200	1340	0.66	149.46	119.28	5.90	0.08	0.35
	B5	1198	1338	0.67	149.63	119.74	5.95	0.07	0.25

Figure 8.1: Beam-column test setup

Figure 8.1 depicts the apparatus employed in the eccentric compression tests on the new high-chromium stainless steel welded I-section beam-columns, including (i) one LVDT, attached to the internal web at the specimen mid-height, to record the lateral deflections about the minor axis, and (ii) two pairs of strain gauges, denoted as SG 1-a and SG 1-b as well as SG 2-a and SG 2-b, attached to the outstand flanges at mid-height, with the detailed locations depicted in Figure 5.1(b) (but with $d_s=10$ mm), to record the longitudinal strains at these positions. The readings recorded from the LVDT and strain gauges were used to calculate the actual initial loading eccentricity e_0 of each beam-column specimen, based on Equation (6.1). Figures 8.2(a) and 8.2(b)

display the failure modes of the tested I-150×100×6 and I-150×120×6 beam-column specimens; all of them exhibited flexural buckling coupled with in-plane bending deformation, both being about the minor axis. The obtained load–mid-height lateral deflection curves for the two series of tested high-chromium stainless steel welded I-section beam-column specimens are shown in Figure 8.3, respectively, while the measured key test results are presented in Table 8.2, including the failure load N_u , the mid-height lateral deflection corresponding to the failure load Δ_u , the actual initial loading eccentricity e_0 , and the first-order elastic ($M_{u,1st,el}$), second-order elastic ($M_{u,2nd,el}$) and second-order inelastic ($M_{u,2nd,inel}$) failure bending moments at the specimen mid-height, as calculated from Equations (8.2)–(8.4), respectively.

$$M_{u,1st,el} = N_u (e_0 + \omega_g) \quad (8.2)$$

$$M_{u,2nd,el} = \frac{M_{u,1st,el}}{(1 - N_u / N_{cr})} \quad (8.3)$$

$$M_{u,2nd,inel} = N_u (e_0 + \omega_g + \Delta_u) \quad (8.4)$$

Table 8.2: Key test results for high-chromium stainless steel welded I-section beam-column specimens

Cross-section	Specimen ID	e_0 (mm)	N_u (kN)	Δ_u (mm)	$M_{u,1st,el}$ (kNm)	$M_{u,2nd,el}$ (kNm)	$M_{u,2nd,inel}$ (kNm)
I-150×100×6	A1	5.0	378.9	11.6	2.0	2.6	6.4
	A2	9.9	311.5	14.1	3.2	4.0	7.6
	A3	20.3	235.7	17.4	4.8	5.7	8.9
	A4	30.0	196.5	20.0	5.9	6.8	9.9
	A5	50.5	134.0	22.4	6.8	7.5	9.8
I-150×120×6	B1	9.6	395.2	14.8	3.9	5.0	9.8
	B2	16.8	318.0	17.9	5.4	6.7	11.1
	B3	26.4	253.3	20.5	6.8	7.9	12.0
	B4	39.9	206.2	22.6	8.3	9.4	13.0
	B5	58.9	162.0	27.3	9.6	10.6	14.0

(a) I-150×100×6 beam-column specimens (from left to right: A1, A2, A3, A4 and A5)

(b) I-150×120×6 beam-column specimens (from left to right: B1, B2, B3, B4, and B5)

Figure 8.2: Failure modes of high-chromium stainless steel welded I-section beam-column specimens under combined compression and minor-axis bending

(a) I-150×100×6 beam-column specimens

(b) I-150×120×6 beam-column specimens

Figure 8.3: Load–mid-height lateral deflection curves for high-chromium stainless steel welded I-section beam-column specimens under combined compression and minor-axis bending

8.3 Numerical modelling

8.3.1 General

In parallel with the testing programme, a numerical modelling programme was conducted, using the nonlinear finite element software ABAQUS (2014). FE models were firstly developed and validated against the test results, and then employed to perform parametric studies to generate further numerical data over a wide range of cross-section dimensions, member lengths and loading combinations.

8.3.2 Development of finite element models

Each high-chromium stainless steel welded I-section beam-column FE model was developed based on the measured cross-section geometric dimensions and member effective length, employing the same modelling assumptions, procedures and techniques as those detailed in Section 6.3.2. Regarding the modelling of the boundary conditions of the beam-column FE models, the top end section was coupled to one reference point, with longitudinal translation and rotation about the minor axis allowed, while the bottom end section was coupled to the other reference point, with only rotation about the minor axis allowed. Both of the top and bottom reference points were located eccentrically to the cross-section minor axis, with the eccentricities given as the corresponding actual initial loading eccentricity e_0 , as listed in Table 8.2, while each of them was longitudinally located at a distance of 70 mm from the corresponding coupled end section. For the incorporation of initial geometric imperfections, two sets of global and local imperfection magnitude combinations, including (i) the measured magnitude combination – $\omega_g + \omega_0$ and (ii) a generalised magnitude combination – $L_e/1000 + t/100$, were employed to factor the obtained initial geometric imperfection distribution patterns. Finally, the developed FE model was solved by means of the geometrically and materially nonlinear

modified Riks method (ABAQUS, 2014), to derive the numerical failure loads, load–mid-height lateral deflection curves and failure modes.

8.3.3 Validation of finite element models

The accuracy of the developed FE models was evaluated through quantitative and graphical comparisons between the obtained test and numerical results. The numerical to test failure load ratios for all the high-chromium stainless steel welded I-section beam-column specimens are presented in Table 8.3, and the results indicated that (i) the numerical failure loads are influenced by the initial geometric imperfection magnitudes and (ii) both the measured and generalised imperfection magnitude combinations result in accurate and consistent predictions of the test failure loads. Comparisons between experimentally and numerically obtained failure modes for two typical high-chromium stainless steel welded I-section beam-column specimens A1 and B4 were then made, as respectively displayed in Figures 8.4 and 8.5, indicating excellent agreement. The FE load–mid-height lateral deflection curves were also shown to well capture their experimental counterparts for the aforementioned two typical beam-column specimens, as shown in Figures 8.6 and 8.7. It can thus be concluded that the test structural responses of the high-chromium stainless steel welded I-section beam-column specimens under combined compression and minor-axis bending moment can be well simulated and the developed FE models are deemed to be reliable and validated.

Figure 8.4: Test and FE failure modes for beam-column specimen A1

Figure 8.5: Test and FE failure modes for beam-column specimen B4

Figure 8.6: Test and FE load–mid-height lateral deflection curves for beam-column specimen A1

Figure 8.7: Test and FE load–mid-height lateral deflection curves for beam-column specimen B4

Table 8.3: Comparison of test and FE failure loads for varying global and local geometric imperfection magnitude combinations

Cross-section	Specimen ID	Finite element N_u / Test N_u	
		$\omega_g + \omega_0$	$L_e/1000 + t/100$
I-150×100×6	A1	1.00	0.96
	A2	1.05	1.01
	A3	1.05	1.00
	A4	0.98	0.95
	A5	1.01	0.96
I-150×120×6	B1	1.03	0.96
	B2	1.04	0.99
	B3	0.98	0.95
	B4	0.99	0.96
	B5	1.01	0.99
	Mean	1.02	0.98
	COV	0.03	0.02

8.3.4 Parametric studies

Having been validated in Section 8.3.3, the developed beam-column FE models were employed in this section to perform parametric studies to generate further numerical data over a broad range of cross-section dimensions, member effective lengths and loading combinations. The modelled I-sections covered all three classes (Class 1, 2 and 3) of non-slender I-sections, according to the slenderness limits given in EN 1993-1-4 (CEN, 2015), and were also categorised as non-slender (non-compact and compact) I-sections, based on the limiting width-to-thickness ratios set out in AISC 27 (AISC, 2013). Specifically, the outer section heights of the modelled I-sections were fixed at 100 mm, while the flange widths were varied from 50 mm to 100 mm, with the plate thicknesses ranging from 3 mm to 10 mm, resulting in a range of cross-section aspect ratios and dimensions being considered. The member effective lengths of the modelled I-sections fell between 540 mm and 3540 mm, leading to a wide spectrum of member non-dimensional slendernesses being accounted for. Moreover,

the initial loading eccentricities (to the minor axis) of the FE models were varied from 0.1 mm to 700 mm, resulting in an extensive range of loading combinations being considered. The geometric dimensions, initial loading eccentricities and load-carrying capacities of the modelled high-chromium stainless steel welded I-section beam-columns are reported in Appendix. The generalised initial global and local geometric imperfection magnitude combination – $L_e/1000$ and $t/100$ was adopted herein. In total, 216 numerical data on the new high-chromium stainless steel welded I-section beam-columns under combined compression and minor-axis bending moment were generated through the parametric studies.

8.4 Evaluation of current design standards and development of new design approach

8.4.1 General

In this section, the experimentally and numerically obtained results were firstly discussed and then used to evaluate the applicability of the design interaction formulae for normal stainless steel welded I-section beam-columns, as given in EN 1993-1-4 (CEN, 2015) and AISC 27 (AISC, 2013), to the new high-chromium stainless steel welded I-section beam-columns. An improved design approach was subsequently developed by overcoming the inherent shortcomings of the codified interaction curves. The following sub-sections 8.4.2–8.4.4 present the results of the graphical and quantitative evaluations for the two considered design standards and the new design approach, with an angle parameter $\theta = \tan^{-1}[(N_{u,pred}/N_R)/(M_{u,pred}/M_R)]$ introduced to reflect the combination of compression and bending moment, as also graphically defined in Figure 8.8, where $N_{u,pred}$ is the design failure load and $M_{u,pred} = N_{u,pred}e_0$ is the design failure moment, while N_R and M_R are the corresponding column flexural buckling resistance and cross-section bending moment resistance; it is worth

noting that the value of θ changes from 0° to 90° when the applied loading varies from isolated bending to isolated compression.

Figure 8.8: Definition of θ

8.4.2 Eurocode EN 1993-1-4

The current Eurocode EN 1993-1-4 (CEN, 2015) specifies supplementary design provisions for stainless steel structures in addition to the general design rules for carbon steel structures set out in EN 1993-1-1 (CEN, 2014). The codified interaction curve for the design of uniform members subject to combined compression and minor-axis bending moment is given by Equation (8.5),

$$\frac{N_{u,pred}}{N_{b,z,Rd}} + k_z \frac{M_{u,pred}}{M_{z,Rd}} \leq 1 \quad (8.5)$$

where $N_{b,z,Rd}$ is the column minor-axis flexural buckling resistance and determined based on the EC3 design buckling curve, as given by Equations (6.2)–(6.5), $M_{z,Rd}$ is

the cross-section minor-axis bending moment resistance, which is taken as the minor-axis plastic moment capacity $M_{pl,z}$ for Class 1 (and 2) welded I-sections but the minor-axis elastic moment capacity $M_{el,z}$ for Class 3 welded I-sections, and k_z is the interaction factor and given by Equation (8.6), considering (i) the interaction between compression and minor-axis bending and (ii) all the second-order effects about the minor principal axis.

$$1.2 \leq k_z = 1 + 2(\bar{\lambda}_z - 0.5) \frac{N_{u,pred}}{N_{b,z,Rd}} \leq 1.2 + 2 \frac{N_{u,pred}}{N_{b,z,Rd}} \quad (8.6)$$

The applicability of the EC3 design interaction curve (CEN, 2015) to the new high-chromium stainless steel welded I-section beam-columns was evaluated, based on the experimentally and numerically obtained data. Table 8.4 lists the mean test (and numerical) to predicted failure load ratios $N_u/N_{u,pred}$ and the corresponding COVs, revealing that the EC3 design interaction curve (CEN, 2015) leads to slightly conservative resistance predictions when used for high-chromium stainless steel Class 1 (and 2) welded I-sections beam-columns, but unduly underestimates the load-carrying capacities of their Class 3 welded I-section counterparts. This is also evident in Figure 8.9, where the test (and numerical) to predicted failure load ratios $N_u/N_{u,pred}$ are plotted against the angle parameter θ . Moreover, it can also be concluded from the graphical evaluation results that (i) the EC3 resistance predictions are accurate when compression is the dominant applied loading (i.e. when the values of θ approach 90°) and (ii) the predicted resistances become increasingly less accurate as the applied loading varies from isolated compression to isolated bending (i.e. as θ varies from 90° to 0°), especially for those high-chromium stainless steel Class 3 welded I-section beam-columns. It is worth noting that the overall accuracy of a design interaction curve is dependent on the end points (i.e. member resistances in isolated bending and isolated compression) and shape (as defined by the interaction factor). Given that the EC3 column minor-axis flexural buckling resistance $N_{b,z,Rd}$ (i.e. the compression end point) is accurate (Sun et al., 2020a, 2020b) and the interaction factor k_z is also

accurate (as reflected by the consistent resistance predictions for high-chromium stainless steel welded I-section beam-columns with the same class of cross-section – see Figure 8.9), the main shortcoming of the EC3 design interaction curve therefore lies in the inaccurate bending end point $M_{z,Rd}$, which is determined without considering material strain hardening and excessively conservative, especially for Class 3 welded I-sections (Sun and Zhao, 2019).

Figure 8.9: Comparisons of test and FE failure loads with predicted failure loads from EN 1993-1-4

Table 8.4: Comparisons of test and FE failure loads with predicted failure loads from EN 1993-1-4

Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,pred}$	
			Mean	COV
Class 1 and 2 section	5	144	1.13	0.08
Class 3 section	5	72	1.37	0.09
Total	10	216	1.21	0.12

8.4.3 American design guide AISC 27

The American design guide AISC 27 (AISC, 2013) adopts a two-stage nonlinear design interaction curve for stainless steel welded I-section beam-columns subject to combined compression and minor-axis bending moment, as given by Equation (8.7), where $N_c = A F_{cr}$ is the column minor-axis flexural buckling resistance, in which F_{cr} is the member critical stress for columns failing by minor-axis flexural buckling and calculated by Equation (8.8), M_{cz} is the cross-section minor-axis bending resistance, which is taken as the minor-axis plastic moment capacity $M_{pl,z} = W_{pl,z} \sigma_{0.2}$ for compact welded I-sections and determined by allowing for partial plasticity based on Equation (8.9) for non-compact welded I-sections, and $\alpha = 1/(1 - N_{u,pred}/N_c)$ is the amplification factor to consider all the second-order effects.

$$\begin{cases} \frac{N_{u,pred}}{N_c} + \frac{8}{9} \frac{\alpha M_{u,pred}}{M_{cz}} \leq 1 & \text{for } \frac{N_{u,pred}}{N_c} \geq 0.2 \\ \frac{N_{u,pred}}{2N_c} + \frac{\alpha M_{u,pred}}{M_{cz}} \leq 1 & \text{for } \frac{N_{u,pred}}{N_c} < 0.2 \end{cases} \quad (8.7)$$

$$F_{cr} = \begin{cases} 0.50 \bar{\lambda}^2 \sigma_{0.2} & \text{for } \bar{\lambda} \leq 1.2 \\ \frac{0.531}{\bar{\lambda}^2} \sigma_{0.2} & \text{for } \bar{\lambda} > 1.2 \end{cases} \quad (8.8)$$

$$M_{cz} = W_{pl,z} \sigma_{0.2} - (W_{pl,z} \sigma_{0.2} - 0.7 W_{el,z} \sigma_{0.2}) \left[\frac{b_f / (2t) - 0.33 \sqrt{E / \sigma_{0.2}}}{0.14 \sqrt{E / \sigma_{0.2}}} \right] \quad (8.9)$$

The AISC design failure loads, as predicted from the interaction curve defined by Equation (8.7), were quantitatively evaluated against the test and numerical failure loads. The quantitative evaluation results are reported in Table 9, where the mean $N_u/N_{u,pred}$ ratio is equal to 1.04 and the corresponding COV is equal to 0.12, revealing that the current AISC design interaction curve yields overall accurate but scattered resistance predictions when applied to the new high-chromium stainless steel welded I-section beam-columns. However, it is worth noting that the overall good design accuracy is actually the compromise of unsafe resistance predictions for high-chromium stainless steel compact welded I-section beam-columns and conservative resistance predictions for high-chromium stainless steel non-compact welded I-

section beam-columns. This is also evident in Figure 8.10, where the graphical evaluation results are presented. Given that the AISC column minor-axis flexural buckling resistance N_c and cross-section minor-axis bending moment resistance M_{cz} have been demonstrated to be accurate by the authors in previous investigations into high-chromium stainless steel welded I-section columns (Sun et al., 2020a, 2020b) and beams (Sun and Zhao, 2019) (i.e. the two end points of the AISC design interaction curve are accurate), the shortcoming of the AISC design interaction curve therefore mainly lies in the inadequate shape.

Figure 8.10: Comparisons of test and FE failure loads with predicted failure loads from AISC 27

Table 8.5: Comparisons of test and FE failure loads with predicted failure loads from AISC 27

Cross-section category	No. of test data	No. of FE data	$N_u/N_{u,pred}$	
			Mean	COV
Compact section	5	120	0.97	0.11
Non-compact section	5	96	1.14	0.10
Total	10	216	1.04	0.12

8.4.4 New design approach

The discussions in Sections 8.4.2 have highlighted that the EC3 design interaction curve has accurate compression end point and shape but suffers from conservative bending end point, which is determined based on the material 0.2% proof stress without considering strain hardening. An improved design method is thus prompted herein through using the EC3 design interaction curve anchored to a new, more accurate bending end point. The continuous strength method (Afshan & Gardner, 2013; Zhao et al., 2017) allows for a rational exploration of strain hardening in calculating the cross-section bending resistances and is thus ideal to be used to determine the new bending end point. The final expression of the new interaction formula is given by Equation (8.10), where $M_{csm,z}$ is the cross-section minor-axis bending resistance as calculated from the CSM. Detailed discussions on the CSM cross-section minor-axis bending moment resistances for stainless steel welded I-sections have been presented in Section 3.4.4.

$$\frac{N_{u,pred}}{N_{b,z,Rd}} + k_z \frac{M_{u,pred}}{M_{csm,z}} \leq 1 \quad (8.10)$$

The resistance predictions from the new design approach, for high-chromium stainless steel welded I-section beam-columns, were compared against the test and numerical failure loads. The graphical and quantitative comparison results are presented in Figure 8.11 and Table 8.6, respectively, revealing that the new design approach leads to safe, accurate and consistent resistance predictions for high-chromium stainless steel welded I-section beam-columns with varying cross-section and member dimensions and loading combinations. The improved design accuracy and consistency of the new approach can also be seen from Figure 8.12, where the codified (EC3 and AISC) and proposed interactive curves are compared with the high-chromium stainless steel welded I-section beam-column test data, showing that

the proposed interactive curve yields a much better representation of the test data points over its codified counterparts.

Figure 8.11: Comparisons of test and FE failure loads with predicted failure loads from new design proposal

Table 8.6: Comparisons of test and FE failure loads with predicted failure loads from new design proposal

Cross-section category*	No. of test data	No. of FE data	$N_u/N_{u,pred}$	
			Mean	COV
Class 1 and 2 section	5	144	1.06	0.06
Class 3 section	5	72	1.07	0.03
Total	10	216	1.06	0.05

* The cross-section category is defined according to EN 1993-1-4.

(a) I-150×100×6 beam-column specimens

(b) I-150×120×6 beam-column specimens

Figure 8.12: Comparisons of test failure loads with EC3, AISC and proposed design interaction curves

8.5 Reliability analysis

The reliability of the new design approach when applied to the new high-chromium stainless steel welded I-section beam-columns was evaluated in this section, according to the requirements and procedures given in EN 1990 (CEN, 2002), as detailed in Section 3.5. A summary of the key calculated statistical parameters for the new design approach is reported in Table 8.7, showing that the required partial factors γ_{M1} is less than the currently adopted value of 1.1 in EN 1993-1-4 (CEN, 2015), and thus demonstrating that the new design proposal satisfy the reliability requirements of EN 1990 (CEN, 2002).

Table 8.7: Reliability analysis results calculated according to EN 1990

	No. of test and FE data	$k_{d,n}$	b	V_δ	V_r	γ_{M1s}
New proposal	226	3.134	1.095	0.044	0.050	0.94

8.6 Concluding remarks

The global stability and load-carrying capacities of the new high-chromium stainless steel welded I-section beam-columns have been investigated through a testing and numerical modelling programme. The testing programme included initial geometric imperfection measurements and minor-axis eccentric compression tests on ten high-chromium stainless steel welded I-section beam-columns with two different cross-section sizes and a range of initial loading eccentricities. The testing programme was supplemented by a numerical modelling programme, including a validation study to validate the developed FE models against the test results and a parametric study to generate further numerical data over a broad range of cross-section dimensions, member lengths and loading combinations. Then, the experimentally and numerically obtained results were adopted to evaluate the applicability of the EC3 and AISC design interaction curves for normal stainless steel welded I-section beam-columns to their high-chromium stainless steel counterparts. The evaluation results indicated

that (i) the EC3 design interaction curve leads to slightly and overly conservative resistance predictions when applied to the new high-chromium stainless steel Class 1 (and 2) and Class 3 welded I-section beam-columns, respectively, mainly due to the conservative bending end moment, which is determined without considering material strain hardening, and (ii) the AISC design interaction curve results in an overall good level of design accuracy, but the majority of the resistance predictions are unsafe-sided for high-chromium stainless steel compact welded I-section-beam columns, owing mainly to the inadequate shape. Finally, a new interaction curve was proposed based on the EC3 interaction curve anchored to a more accurate CSM bending end point, and shown to offer significantly improved design accuracy and consistency than its EC3 and AISC counterparts.

CHAPTER 9

CONCLUSIONS AND SUGGESTIONS FOR FUTURE RESEARCH

9.1 Conclusions

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

High-performance steels, including stainless steels and high strength steels, have been gaining increasing popularity in civil engineering, owing to their appealing sustainable and mechanical advantages. Given the higher material prices of high-performance steels in comparison with those of the conventional mild steels, structural design efficiency is of primary concern. However, most of the provisions given in the existing design standards for stainless steel and high strength steel welded I-section components were developed by directly mirroring those for their conventional mild steel counterparts, with a low level of design efficiency. Therefore, investigations of high-performance steel welded I-section components and development of efficient design rules are the objectives of the present thesis.

The local buckling behaviour and resistances of welded I-section stub columns and beams made of the new grade EN 1.4420 high-chromium austenitic stainless steel have been experimentally and numerically investigated in Chapter 3 (Sun & Zhao, 2019). The world's first set of tests on high-chromium stainless steel materials and

cross-sections have been reported. On the basis of the measured data, a new predictive model that is capable of predicting the membrane residual stress distributions and amplitudes in high-chromium stainless steel welded I-sections has been proposed. The applicability of the relevant codified slenderness limits and local buckling design rules provided in EN 1993-1-4 (CEN, 2015) and AISC 27 (AISC, 2013) to the new high-chromium stainless steel welded I-section stub columns and beams has been evaluated. The codified slenderness limits were found to be applicable to the new high-chromium stainless steel welded I-sections, while the codified local buckling design rules were shown to yield conservative and scattered cross-section resistance predictions, owing principally to the lack of consideration of the favourable material strain hardening of high-chromium stainless steel and element interaction within I-sections. The accuracy of the continuous strength method (CSM) (Afshan & Gardner, 2013; Zhao et al., 2017) for the new high-chromium stainless steel welded I-sections has also been assessed. The CSM accounts for the beneficial effects of material strain hardening and element interaction in calculating cross-section compression and bending resistances and was shown to yield substantially improved design accuracy and consistency over the existing design standards (CEN, 2015; AISC, 2013).

Chapter 4 reports the experimental and numerical studies of the local stability and load-carrying capacities of S690 high strength steel welded I-sections under isolated loading conditions (Sun et al., 2019a, 2019b). A comprehensive testing programme, including material tensile coupon tests, membrane residual stress measurements, sixteen stub column tests and twelve in-plane four-point bending tests, with six for each principal axis, has been conducted, followed by a numerical modelling programme. A new predictive model for membrane residual stress distributions and amplitudes in S690 high strength steel welded I-sections has been proposed. The accuracy of the codified slenderness limits and local buckling design rules for S690 high strength steel welded I-section stub columns and beams, as given in EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016), has

been assessed for the first time. The assessment results generally indicated that (i) most of the codified slenderness limits are safe and accurate when applied to S690 high strength steel welded I-sections in pure compression and pure bending, and (ii) all the three design codes standards yield accurate and consistent cross-section compression resistance predictions for S690 high strength steel welded I-section stub columns, but lead to conservative and scattered predicted bending moment resistances for S690 high strength steel welded I-section beams.

In Chapter 5, the local buckling behaviour and resistances of S690 high strength steel welded I-sections under combined compression and bending moment have been studied for the first time around the world (Sun et al., 2021c). This chapter reports the first set of tests on S690 high strength steel welded I-section stub columns subjected to both major-axis and minor-axis combined loading, followed by numerical modelling. On the basis of the obtained test and numerical data, the accuracy of the codified design interaction curves, as given in EN 1993-1-12 (CEN, 2007), AS 4100 (AS, 2016) and ANSI/AISC 360-16 (AISC, 2016), for S690 high strength steel welded I-sections under combined loading has been evaluated for the first time. The evaluation results revealed that (i) EN 1993-1-12 (CEN, 2007) yields accurate resistance predictions for S690 high strength steel Class 1 and 2 welded I-sections under both major-axis and minor-axis combined loading, but underestimates the load-carrying capacities of their Class 3 counterparts, (ii) AS 4100 (AS, 2016) leads to excessive conservatism for S690 high strength steel non-slender welded I-sections under combined loading, and (iii) ANSI/AISC 360-16 (AISC, 2016) can well predict the cross-section resistances for S690 high strength steel non-slender welded I-sections under major-axis combined loading, but yields conservative resistance predictions for those under minor-axis combined loading. These research findings will serve as a basis for future revisions of EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016).

Complementary to these studies of the local buckling behaviour of high-performance steel welded I-sections, investigations into the member instability of high-performance steel welded I-section columns (reported in Chapters 6 and 7) and beam-columns (reported in Chapter 8) have also been conducted. Experimental and numerical studies of the flexural buckling behaviour and local–flexural interactive buckling behaviour of the new high-chromium austenitic stainless steel welded I-section columns have been presented in Chapter 6 (Sun et al., 2020a, 2020b). It begins with a well-designed testing programme, which adopted four high-chromium stainless steel welded I-sections and included 20 pin-ended column tests. It is worth highlighting that these column tests on high-chromium stainless steel welded I-sections were the first to be reported. The testing programme was followed by a numerical modelling programme. The obtained test and numerical data were used to assess the applicability of the relevant design rules for normal stainless steel welded I-section columns, as set out in EN 1993-1-4 (CEN, 2015) and AISC 360 (AISC, 2013), to the new high-chromium stainless steel welded I-section columns. It has been verified that both the two sets of codified design rules can be safely extended to cover the design of the new high-chromium stainless steel welded I-section columns failing by flexural buckling and local–flexural interactive buckling.

In Chapter 7, experimental and numerical investigations of the member instability of S690 high strength steel welded I-section columns have been performed (Sun et al., 2020c, 2021a). The overall experimental programme includes two parts: (i) ten pin-ended compression tests on S690 high strength steel non-slender welded I-section columns susceptible to flexural buckling and (ii) ten pin-ended compression tests on S690 high strength steel slender welded I-section columns susceptible to local–flexural interactive buckling, which are the first such tests around the world. On the basis of the obtained test and numerical data, the accuracy of the codified buckling curves, as given in EN 1993-1-2 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016), for S690 high strength steel non-slender welded I-section

columns susceptible to flexural buckling was assessed, indicating inaccuracy. A new buckling curve has been proposed, leading to a higher level of design accuracy over the codified buckling curves. The codified design rules for S690 high strength steel slender welded I-section columns susceptible to local–flexural interactive buckling were also found to be inaccurate. A new design approach has been developed through the use of the newly proposed buckling curve (for considering flexural buckling) combined with the EC3 effective width method (for considering local buckling), and shown to yield more accurate interactive buckling resistances than the codified design rules. These research findings and new design approaches will serve as a basis for future revisions of EN 1993-1-12 (CEN, 2007), ANSI/AISC 360-16 (AISC, 2016) and AS 4100 (AS, 2016).

Chapter 8 reports experimental and numerical investigations of the global buckling behaviour of the new high-chromium stainless steel welded I-section beam-columns for the first time around the world (Sun et al., 2021b). A testing programme, including ten high-chromium stainless steel welded I-section beam-column tests, has been conducted, followed by a numerical modelling programme. The test and numerical data have been adopted to examine the applicability of the design interaction curves for normal stainless steel welded I-section beam-columns, as given in EN 1993-1-4 (CEN, 2015) and AISC 27 (AISC, 2013), to their high-chromium counterparts. It was found that the Eurocode design interaction curve leads to conservative resistance predictions, owing mainly to the conservative bending end point of the interaction curve, which is calculated without considering material strain hardening. The interaction curve specified in the American specification was shown to result in overall good design accuracy, but with many unsafe resistance predictions, which can be principally attributed to the inadequate shape of the interaction curve. Finally, an improved design interaction curve has been proposed through employing the Eurocode design interaction curve anchored to a more accurate bending end point,

which is determined with rational consideration of strain hardening by the CSM (Afshan & Gardner, 2013; Zhao et al., 2017). The proposed interaction curve was shown to yield accurate, consistent and safe resistance predictions for the new high-chromium stainless steel welded I-section beam-columns.

Overall, in-depth experimental and numerical investigations into the local and global buckling behaviour of high-performance steel welded I-section structural components under various types of loading (e.g., pure compression, pure bending and combined compression and bending) have been conducted and reported in this thesis. The corresponding codified design rules for each type of the high-performance steel welded I-section structural components have been fully discussed and evaluated, with their shortcomings highlighted. New design methods for various high-performance steel welded I-section structural components have been proposed and shown to offer improved levels of design accuracy and consistency. The reliability of the new design proposals has been confirmed by means of statistical analyses according to EN 1990 (CEN, 2002). These research findings and new design proposals will serve as a basis for future revisions of international design standards for stainless steel and high strength steel structures.

9.2 Suggestions for future research

Building on the present research, suggestions for future research on high-performance steel welded I-section structures are given herein. Firstly, investigations into the behaviour and load-carrying capacities of high-performance steel welded I-section components under other types of loading beyond those considered in the present thesis are required, for example, the shear buckling behaviour of high-performance steel welded I-section plate girders under shear force and the global buckling behaviour of high-performance steel welded I-section beam-columns under moment gradients.

The global instability of high-performance steel welded I-section columns and beam-columns is influenced by the boundary conditions. In this thesis, pin-ended boundary conditions were applied onto both ends of each column or beam-column specimen. Future research could be conducted to investigate the global buckling behaviour of high-performance steel welded I-section columns and beam-columns with other boundary conditions, e.g., pinned-fixed and fixed-fixed end restraints.

While this thesis investigates the structural behaviour of high-performance steel welded I-sections, future research is required to cover other cross-section profiles, e.g., channel sections, T-sections and angle sections. For non-doubly symmetric sections with low torsional constant, torsional-related buckling would be triggered and special design attentions should then be given to torsional buckling and flexural-torsional buckling.

Finally, the structural behaviour of high-performance welded I-section components in fire (at elevated temperatures) and after exposure to fire (after exposure to elevated temperatures) requires investigations. Laboratory tests and numerical simulations could be performed to investigate their fire and post-fire behaviour and resistances under various types of loading, followed by the development of efficient design rules.

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APPENDIX

Table A.1: Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled S690 high strength steel welded I-section stub columns under combined loading

(a) Major-axis combined loading					
L (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
300	100	50	4.0	3	511.1
300	100	50	4.0	19	379.8
300	100	50	4.0	35	302.7
300	100	50	4.0	80	192.8
300	100	50	4.0	200	97.4
300	100	50	4.5	11	505.0
300	100	50	4.5	28	392.1
300	100	50	4.5	51	301.6
300	100	50	4.5	100	197.8
300	100	50	4.5	270	84.5
300	100	50	5.0	50	362.3
300	100	50	5.0	70	286.6
300	100	50	5.0	100	224.5
300	100	50	5.0	200	125.8
300	100	50	5.0	290	89.1
300	100	60	4.5	500	50.5
300	100	60	4.5	27	439.7
300	100	60	4.5	67	286.9
300	100	60	4.5	110	204.3
300	100	60	4.5	300	84.2
300	100	60	4.8	3	690.2
300	100	60	4.8	22	505.7
300	100	60	4.8	44	387.0
300	100	60	4.8	77	284.5
300	100	60	4.8	800	34.6
300	100	60	5.2	1	783.6
300	100	60	5.2	47	413.2
300	100	60	5.2	66	344.5
300	100	60	5.2	130	209.0
300	100	60	5.2	1000	30.4
300	100	70	5.0	5	757.8
300	100	70	5.0	12	670.6
300	100	70	5.0	48	416.2

Table A.1 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled S690 high strength steel welded I-section stub columns under combined loading

L (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
300	100	70	5.0	116	239.8
300	100	70	5.0	400	78.2
300	100	70	5.5	8	802.1
300	100	70	5.5	17	693.1
300	100	70	5.5	38	519.2
300	100	70	5.5	90	319.2
300	100	70	5.5	240	141.2
300	100	70	6.5	7	992.7
300	100	70	6.5	28	709.1
300	100	70	6.5	77	433.5
300	100	70	6.5	170	233.0
300	100	70	6.5	700	60.1
300	100	85	5.0	53	438.6
300	100	85	5.0	85	330.8
300	100	85	5.0	116	264.0
300	100	85	5.0	173	196.4
300	100	85	5.0	400	89.5
300	100	85	7.0	2	1292.9
300	100	85	7.0	15	1041.7
300	100	85	7.0	40	752.3
300	100	85	7.0	80	516.7
300	100	85	7.0	150	319.7
300	100	85	8.0	4	1466.7
300	100	85	8.0	27	1045.9
300	100	85	8.0	59	735.0
300	100	85	8.0	120	447.7
300	100	85	8.0	400	146.3
300	100	100	5.8	9	1010.1
300	100	100	5.8	17	883.7
300	100	100	5.8	38	663.6
300	100	100	5.8	73	468.4
300	100	100	5.8	140	298.0
300	100	100	8.0	2	1622.8
300	100	100	8.0	10	1410.0
300	100	100	8.0	25	1125.4
300	100	100	8.0	50	839.7
300	100	100	8.0	100	552.9
300	100	100	12.0	1	2201.3
300	100	100	12.0	7	1990.1

Table A.1 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled S690 high strength steel welded I-section stub columns under combined loading

L (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
300	100	100	12.0	35	1338.2
300	100	100	12.0	70	939.3
300	100	100	12.0	200	448.5
(b) Major-axis combined loading					
L (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
300	100	50	4.0	9	260.6
300	100	50	4.0	12	213.1
300	100	50	4.0	15	183.7
300	100	50	4.0	20	142.0
300	100	50	4.0	60	54.8
300	100	50	4.3	2	477.0
300	100	50	4.3	6	368.3
300	100	50	4.3	12	237.8
300	100	50	4.3	27	129.4
300	100	50	4.3	10	282.5
300	100	50	6.6	3	705.5
300	100	50	6.6	7	548.6
300	100	50	6.6	14	363.7
300	100	50	6.6	21	244.7
300	100	50	6.6	55	112.4
300	100	60	4.0	2	518.8
300	100	60	4.0	9	359.2
300	100	60	4.0	18	230.7
300	100	60	4.0	30	148.2
300	100	60	4.0	50	93.8
300	100	60	5.0	175	38.3
300	100	60	5.0	30	197.9
300	100	60	5.0	7	486.5
300	100	60	5.0	5	551.4
300	100	60	5.0	2	625.7
300	100	60	8.0	3	1029.9
300	100	60	8.0	7	861.0
300	100	60	8.0	11	651.0
300	100	60	8.0	22	384.9
300	100	60	8.0	80	126.5
300	100	70	4.5	5	589.5
300	100	70	4.5	9	504.3
300	100	70	4.5	13	424.7

Table A.1 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled S690 high strength steel welded I-section stub columns under combined loading

L (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
300	100	70	4.5	18	355.9
300	100	70	4.5	46	161.0
300	100	70	6.0	7	713.3
300	100	70	6.0	17	483.7
300	100	70	6.0	23	378.5
300	100	70	6.0	28	315.4
300	100	70	6.0	50	199.8
300	100	70	12.0	4	1569.7
300	100	70	12.0	5	1496.1
300	100	70	12.0	19	856.7
300	100	70	12.0	36	533.5
300	100	70	12.0	60	342.0
300	100	85	5.0	5	786.2
300	100	85	5.0	8	731.0
300	100	85	5.0	10	680.4
300	100	85	5.0	14	604.5
300	100	85	5.0	88	144.2
300	100	85	7.0	3	1152.8
300	100	85	7.0	12	882.5
300	100	85	7.0	20	688.4
300	100	85	7.0	44	377.6
300	100	85	7.0	195	89.1
300	100	85	10.0	2	1720.4
300	100	85	10.0	8	1447.8
300	100	85	10.0	17	1120.9
300	100	85	10.0	33	697.0
300	100	85	10.0	100	258.7
300	100	100	6.0	4	1115.0
300	100	100	6.0	15	861.7
300	100	100	6.0	25	678.8
300	100	100	6.0	35	533.6
300	100	100	6.0	50	392.4
300	100	100	8.0	1	1559.9
300	100	100	8.0	10	1221.3
300	100	100	8.0	50	520.9
300	100	100	8.0	27	830.5
300	100	100	8.0	100	270.8
300	100	100	9.0	5	1639.1
300	100	100	9.0	15	1263.1
300	100	100	9.0	23	1042.6
300	100	100	9.0	200	171.1
300	100	100	9.0	80	379.0

Table A.2: Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled high-chromium stainless steel welded I-section beam-columns

L_e (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
580	100	50	3.0	2	99.6
580	100	50	3.0	5	76.5
580	100	50	3.0	12	51.0
580	100	50	3.0	23	32.2
580	100	50	3.0	41	20.6
580	100	50	3.0	78	12.2
840	100	50	3.0	7	50.4
840	100	50	3.0	9	44.7
840	100	50	3.0	12	40.2
840	100	50	3.0	15	35.9
840	100	50	3.0	23	28.3
840	100	50	3.0	29	24.5
1270	100	50	3.0	3	34.3
1270	100	50	3.0	9	27.0
1270	100	50	3.0	17	21.8
1270	100	50	3.0	31	16.6
1270	100	50	3.0	45	13.5
1270	100	50	3.0	72	10.1
1500	100	50	3.0	5	24.7
1500	100	50	3.0	12	20.0
1500	100	50	3.0	26	15.2
1500	100	50	3.0	41	12.3
1500	100	50	3.0	50	11.1
1500	100	50	3.0	78	8.6
580	100	50	3.5	5	95.9
580	100	50	3.5	12	64.7
580	100	50	3.5	27	39.5
580	100	50	3.5	39	29.9
580	100	50	3.5	85	16.0
580	100	50	3.5	130	11.3
860	100	50	3.5	3	74.3
860	100	50	3.5	8	57.0
860	100	50	3.5	15	42.3
860	100	50	3.5	43	22.4
860	100	50	3.5	73	15.5
860	100	50	3.5	145	9.0
1030	100	50	3.5	19	31.9
1030	100	50	3.5	27	26.5
1030	100	50	3.5	38	21.6

Table A.2 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled high-chromium stainless steel welded I-section beam-columns

L_e (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
1030	100	50	3.5	52	17.7
1030	100	50	3.5	80	13.1
1030	100	50	3.5	195	6.6
1340	100	50	3.5	0.5	48.6
1340	100	50	3.5	3	40.1
1340	100	50	3.5	10	29.9
1340	100	50	3.5	15	26.2
1340	100	50	3.5	75	11.6
1340	100	50	3.5	800	2.0
540	100	50	6.0	8	149.3
540	100	50	6.0	11	127.0
540	100	50	6.0	13	115.7
540	100	50	6.0	17	98.2
540	100	50	6.0	33	62.0
540	100	50	6.0	77	32.3
950	100	50	6.0	4	108.8
950	100	50	6.0	15	68.6
950	100	50	6.0	25	52.6
950	100	50	6.0	35	43.1
950	100	50	6.0	75	25.7
950	100	50	6.0	250	10.3
1250	100	50	6.0	3	86.6
1250	100	50	6.0	12	60.7
1250	100	50	6.0	25	44.3
1250	100	50	6.0	45	32.0
1250	100	50	6.0	80	22.0
1250	100	50	6.0	170	12.7
1550	100	50	6.0	0.1	74.7
1550	100	50	6.0	13	44.8
1550	100	50	6.0	29	32.9
1550	100	50	6.0	60	22.6
1550	100	50	6.0	85	18.3
1550	100	50	6.0	200	10.1
820	100	75	3.7	4	180.9
820	100	75	3.7	10	137.7
820	100	75	3.7	18	102.9
820	100	75	3.7	27	80.9
820	100	75	3.7	42	57.7
820	100	75	3.7	70	39.0

Table A.2 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled high-chromium stainless steel welded I-section beam-columns

L_e (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
1260	100	75	3.7	2	133.3
1260	100	75	3.7	9	100.4
1260	100	75	3.7	17	79.1
1260	100	75	3.7	31	57.9
1260	100	75	3.7	49	45.0
1260	100	75	3.7	81	31.7
1850	100	75	3.7	7	66.8
1850	100	75	3.7	11	59.8
1850	100	75	3.7	19	50.3
1850	100	75	3.7	33	40.2
1850	100	75	3.7	55	31.0
1850	100	75	3.7	76	25.6
2240	100	75	3.7	5	52.7
2240	100	75	3.7	14	42.7
2240	100	75	3.7	28	34.5
2240	100	75	3.7	42	29.3
2240	100	75	3.7	66	23.6
2240	100	75	3.7	86	20.4
1200	100	75	4.5	8	130.7
1200	100	75	4.5	18	96.7
1200	100	75	4.5	32	71.8
1200	100	75	4.5	68	45.0
1200	100	75	4.5	123	28.8
1200	100	75	4.5	460	9.9
1520	100	75	4.5	11	92.2
1520	100	75	4.5	25	67.2
1520	100	75	4.5	41	52.2
1520	100	75	4.5	75	36.0
1520	100	75	4.5	135	24.0
1520	100	75	4.5	400	10.2
1900	100	75	4.5	9	74.0
1900	100	75	4.5	20	58.3
1900	100	75	4.5	60	35.1
1900	100	75	4.5	90	27.6
1900	100	75	4.5	120	22.9
1900	100	75	4.5	300	11.8
2300	100	75	4.5	5	61.6
2300	100	75	4.5	12	52.1
2300	100	75	4.5	25	42.3

Table A.2 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled high-chromium stainless steel welded I-section beam-columns

L_e (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
2300	100	75	4.5	40	35.4
2300	100	75	4.5	80	25.3
2300	100	75	4.5	230	13.1
640	100	75	8.0	8	396.2
640	100	75	8.0	12	336.2
640	100	75	8.0	16	290.7
640	100	75	8.0	20	255.1
640	100	75	8.0	50	135.1
640	100	75	8.0	90	84.0
1240	100	75	8.0	10	209.6
1240	100	75	8.0	15	181.1
1240	100	75	8.0	35	118.8
1240	100	75	8.0	50	95.3
1240	100	75	8.0	80	69.2
1240	100	75	8.0	400	20.1
1740	100	75	8.0	2	188.5
1740	100	75	8.0	7	156.6
1740	100	75	8.0	13	133.0
1740	100	75	8.0	20	114.8
1740	100	75	8.0	40	84.1
1740	100	75	8.0	140	38.8
2240	100	75	8.0	4	117.8
2240	100	75	8.0	15	91.2
2240	100	75	8.0	35	68.8
2240	100	75	8.0	60	53.9
2240	100	75	8.0	130	34.7
2240	100	75	8.0	350	17.5
1340	100	100	5.0	6	261.1
1340	100	100	5.0	20	178.9
1340	100	100	5.0	40	120.8
1340	100	100	5.0	60	94.8
1340	100	100	5.0	85	74.0
1340	100	100	5.0	120	56.3
2040	100	100	5.0	9	155.1
2040	100	100	5.0	17	129.2
2040	100	100	5.0	24	114.0
2040	100	100	5.0	38	93.1
2040	100	100	5.0	44	86.5
2040	100	100	5.0	47	83.5

Table A.2 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled high-chromium stainless steel welded I-section beam-columns

L_e (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
2820	100	100	5.0	8	101.3
2820	100	100	5.0	14	90.4
2820	100	100	5.0	27	75.0
2820	100	100	5.0	48	60.3
2820	100	100	5.0	120	37.6
2820	100	100	5.0	200	27.4
3340	100	100	5.0	11	73.7
3340	100	100	5.0	21	64.0
3340	100	100	5.0	33	56.3
3340	100	100	5.0	57	46.3
3340	100	100	5.0	140	29.9
3340	100	100	5.0	285	19.4
740	100	100	6.5	5	536.3
740	100	100	6.5	15	401.9
740	100	100	6.5	30	275.1
740	100	100	6.5	45	206.6
740	100	100	6.5	90	118.6
740	100	100	6.5	400	32.8
1640	100	100	6.5	1	347.1
1640	100	100	6.5	10	248.5
1640	100	100	6.5	30	163.8
1640	100	100	6.5	60	110.7
1640	100	100	6.5	120	69.4
1640	100	100	6.5	500	22.3
2640	100	100	6.5	3	164.7
2640	100	100	6.5	8	144.6
2640	100	100	6.5	18	120.0
2640	100	100	6.5	35	95.9
2640	100	100	6.5	70	69.9
2640	100	100	6.5	300	27.7
3540	100	100	6.5	13	85.2
3540	100	100	6.5	22	75.8
3540	100	100	6.5	38	65.0
3540	100	100	6.5	80	49.0
3540	100	100	6.5	115	41.2
3540	100	100	6.5	700	12.8
1040	100	100	10.0	16	478.9
1040	100	100	10.0	18	455.0
1040	100	100	10.0	60	220.2

Table A.2 (continued): Geometric dimensions, initial loading eccentricities and load-carrying capacities of modelled high-chromium stainless steel welded I-section beam-columns

L_e (mm)	h (mm)	b_f (mm)	t (mm)	e_0 (mm)	N_u (kN)
1040	100	100	10.0	90	162.7
1040	100	100	10.0	170	97.1
1040	100	100	10.0	350	55.8
1490	100	100	10.0	10	417.4
1490	100	100	10.0	28	281.1
1490	100	100	10.0	49	205.5
1490	100	100	10.0	77	152.8
1490	100	100	10.0	105	122.5
1490	100	100	10.0	500	36.2
2140	100	100	10.0	5	318.4
2140	100	100	10.0	15	250.2
2140	100	100	10.0	35	182.9
2140	100	100	10.0	65	133.4
2140	100	100	10.0	95	106.2
2140	100	100	10.0	200	63.7
3150	100	100	10.0	0.1	210.8
3150	100	100	10.0	14	152.5
3150	100	100	10.0	28	127.8
3150	100	100	10.0	55	99.8
3150	100	100	10.0	130	65.8
3150	100	100	10.0	700	20.8