

Behaviour and design of high strength steel structural elements and frames

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For my parents:

Mrs Sanmei Zhang and Mr Hongjin Zhu

Declaration of originality

I confirm that this thesis is my own work and that any material from published or unpublished work from others is appropriately referenced. This thesis is submitted to Imperial College London for the degree of Doctor of Philosophy and has not been submitted for a degree or any other qualification at another university.

Yufei Zhu
March 2023, London

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Abstract

The behaviour and design of high strength steel (HSS) welded I-section structural elements and frames subjected to different loading configurations are studied in this thesis. The thesis is intended to evaluate and improve the current Eurocode 3 (EC3) design provisions for HSS structures, and to facilitate the deformation-based advanced design of HSS welded I-section structural elements by means of second-order inelastic analysis with strain limits. Normal strength steel (NSS) counterparts are also studied for comparison purposes.

A novel residual stress model for welded I-sections made of varying steel grades has been firstly developed and calibrated based on a large data pool of residual stress measurements collected from literature. This model was employed in subsequent numerical studies throughout the thesis. A comprehensive experimental programme on HSS welded I-section structural elements and frames was then carefully conducted. At the member level, a total of six three-point bending tests and five column flexural buckling tests were performed, while at the system-level, eight fixed-base single-storey HSS frames were tested in-plane under different combinations of horizontal and vertical loads. Digital image correlation (DIC) was used in the experimental programme to trace the spread of plasticity at critical cross-sections as well as the full-range deformation-field of the frame specimens. The test results, together with existing test data on HSS welded I-section members collected from literature, were used for the validation of finite element (FE) models. Numerical parametric studies were then carried out using the validated FE models to generate extensive structural performance data. Based on the experimental and numerical data, modified EC3 design rules for individual members under compression and combined loading were developed, showing improved levels of accuracy and consistency relative to the current EC3 design rules. In addition, the test results of the frames highlighted the potential for extending plastic design to HSS structures, which is currently prohibited in EC3. Finally, an advanced deformation-based design method by second-order inelastic analysis with strain limits was extended to HSS welded I-section structural members and has been shown to provide superior resistance predictions compared with the current EC3 design methods.

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List of Publications

The journal articles, based on the work presented in this thesis, are listed in chronological order as follows:

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Zhu, Y., Yun, X. & Gardner, L. (2023) Numerical modelling and design of normal and high strength steel non-slender welded I-section beam-columns. *Thin-walled Structures* 186: 110654.

Nomenclature

A	Gross cross-sectional area
a, b, c, d	Residual stress distribution parameters
A_{eff}	Effective cross-sectional area
A_f	Total areas of the flange plates
A_w	Total areas of the web plates
B	Width of section
b	Mean value correction factor
b_f	Clear (flat) width of the flange plate
C_1	CSM limiting strain coefficient
C_m	Equivalent uniform moment factor
COV	Coefficient of variation
CSM	Continuous strength method
DIC	Digital image correlation
E	Young's modulus
e_0	Initial loading eccentricity
E_f	Young's modulus of the flange plate
e_g	Total column global imperfection
E_{sh}	Strain hardening modulus
E_w	Young's modulus of the web plate
$F_{1\text{st_hinge}}$	First order first plastic hinge load
$F_{1\text{st_pl}}$	First order plastic load
F_{cr}	Elastic critical load
F_d	Design load applied to the frame specimen

F_u	Experimental ultimate load
f_u	Ultimate tensile strength
f_y	Yield strength
$f_{y,0.2}$	Yield strength (0.2% proof stress)
$f_{y,a}$	Weighted average yield strength
$f_{y,f}$	Yield strength of the flange plates
$f_{y,w}$	Yield strength of the web plate
H	Outer section depth of cross-section; Horizontal load applied to the frame specimen
h	Depth of web (including welds); Height of the column of the frame specimen
$H_{el,w}$	Height of the elastic portion of the web plate
HSS	High strength steel
h_w	Clear depth (flat) of web
HYB	Hybrid material
I	Second moment of area
k	Interaction factor
$k_{d,n}$	Design fractile factor for n data points
k_σ	Elastic buckling factor
L	Length of specimen
$L_{b,cs}$	Elastic local buckling half-wavelength of cross-section in compression
L_{cr}	Critical effective buckling length of the column specimens
L_e	Effective length of specimen
L_s	Shear span
L_{span}	Length of beam specimen
LVDT	Linear variable displacement transducer
M	Bending moment
M_{AISC}	Bending moment predicted according to AISC 360-16 (2016)

M_c	Bending moment at beam-to-column connection
M_{cb}	Bending moment at column-base connection
M_{csm}	Bending moment predicted using the CSM
M_{EC3}	Bending moment predicted according to EN 1993-1-1 (2005, 2019)
M_{Ed}	Applied design bending moment
M_{eff}	Effective bending moment capacity
M_{el}	Elastic moment capacity
M_{ep}	Elasto-plastic moment capacity
M_{pl}	Plastic moment capacity
M_{pred}	Bending moment capacities predicted using considered design method
M_{shell}	Bending moment obtained from shell FE models
M_u	Ultimate bending moment capacity
n	Ratio of design axial force to design value of buckling resistance of a member in compression or number of test and FE data
N_b	Flexural buckling resistance
N_{cr}	Elastic (Euler) buckling load
N_{Ed}	Applied design axial-load
N_{pl}	Plastic squash load
N_{pred}	Flexural buckling load predicted using considered design method
N_{shell}	Flexural buckling load obtained from shell FE models
NSS	Normal strength steel
N_u	Ultimate axial load (unfactored)
$N_{u,pred}$	Ultimate capacity predicted using considered design method
PH	Plastic hinge
R	Rotation capacity (determined according to M_{pl})
$R_{0.95}$	Rotation capacity (determined according to $0.95M_{pl}$)
r_e	Experimental (or numerical) resistance

RHS	Rectangular hollow section
R_{pred}	Radial distance from the origin to the datapoint predicted using the considered design method
R_{req}	Rotation capacity requirement
R_{shell}	Radial distance from the origin to the datapoint of the shell FE results
r_t	Theoretical (predicted) resistance
R_u	Ultimate resistance/capacity
SG	Strain gauge
SHS	Square hollow section
SP	String potentiometer
t	Thickness
t_f	Flange thickness
t_w	Web thickness
t_{weld}	Weld leg length
V	Vertical load applied to the frame specimen
V_A	COV of cross-sectional area
V_E	COV of Young's modulus
V_{Ed}	Design shear force
V_{fy}	COV of yield strength
$V_{\text{pl,Rd}}$	Plastic shear resistance
V_r	Combined COV accounting for variability of resistance model and basic variables
V_{rt}	Combined COV of the geometric and material parameters
V_u	Ultimate shear capacity
V_δ	Coefficient of variation of test or FE results relative to resistance model
W_{eff}	Effective section modulus
W_{el}	Elastic section modulus
$W_{\text{el,f}}$	Elastic section modulus associated with the flange plates

$W_{el,w}$	Elastic section modulus associated with the elastic portion of the web plate
W_{ep}	Elasto-plastic section modulus
w_g	Measured initial global geometric imperfection
W_{pl}	Plastic section modulus
$W_{pl,w}$	Plastic section modulus associated with the plastic portion of the web plate
α	Imperfection factor
α_{amp}	Amplification factor that accounts for the second-order effects in beam-column members
α_{cr}	Elastic critical load factor
α_{peak}	Peak load factor
α_u	Ultimate load factor
β_{ep}	Interpolation factor for semi-compact cross-sections in bending
γ_{M0}	Partial safety factor for cross-section resistance
γ_{M1}	Partial safety factor for member buckling resistance
γ_{M1}^*	Partial safety factor for member buckling resistance
Δ	Lateral deflection at mid-height of specimens
δ_{base}	Horizontal displacement at the column base of the frame specimen
δ_f	Local imperfection amplitude of the flange
δ_{mid}	Vertical displacement at the mid-span of the beam of the frame specimen
δ_u	End shortening of stub columns at ultimate load
δ_w	Local imperfection amplitude of the web
ε	Strain
ε_{csm}	CSM strain that a cross-section can sustain
ε_{Ed}	Design value of compressive strain
$\varepsilon_{Ed,av}$	Average design value of compressive strain
ε_f	Strain at fracture measured over the standard gauge length
ε_{max}	Measured strain at the maximum compressive fibre

$\varepsilon_{\min}$	Measured maximum tensile or minimum compressive strain at the other extreme fibre
ε_{sh}	Strain hardening strain
ε_u	Strain at ultimate tensile stress
ε_y	Yield strain
θ	Angle parameter
θ_b	Rotation at the beam end
$\theta_{b,flex}$	Rotation resulting from beam flexure
θ_c	Rotation of beam-to-column connection
θ_{cb}	Rotation of column-base connection
θ_{col}	Rotation at the column end
θ_{col+pz}	Rotation of the strengthening plate
θ_{pz}	Rotation of the panel zone
κ	Curvature under bending moment
κ_{pl}	Elastic component of curvature at M_{pl}
κ_{rot}	Curvature when mid-span moment falls below M_{pl} on the descending branch
$\bar{\lambda}$	Non-dimensional member slenderness
$\bar{\lambda}_{LT}$	Non-dimensional lateral torsional buckling slenderness
$\bar{\lambda}_p, \bar{\lambda}_{p,cs}$	Cross-section slenderness
$\bar{\lambda}_{p,f}$	Flange plate slenderness
$\bar{\lambda}_{p,w}$	Web plate slenderness
$\bar{\lambda}_{0,EC3}$	Plateau length of the buckling curve according to EN 1993-1-1 (2005, 2019)
ν	Poisson's ratio
ρ_{csm}	Reduction factor applied to CSM strain limit to consider effect from high shear force
σ	Stress
$\sigma_{cr}, \sigma_{cr,cs}$	Elastic local buckling stress of the cross-section
$\sigma_{cr,f}$	Elastic buckling stress of the flange plate

$\sigma_{cr,w}$	Elastic buckling stress of the web plate
σ_r	Measured residual stresses
$\sigma_{r,fc}$	The maximum compressive residual stresses in the flanges
$\sigma_{r,ft}$	The maximum tensile residual stresses in the flanges
$\sigma_{r,wc}$	The maximum compressive residual stresses in the web
$\sigma_{r,wt}$	The maximum tensile residual stresses in the web
φ	Mid-span rotation
ϕ_c	Resistance factor for member buckling resistance according to AISC 360-16 (2016)
φ_{max}	The maximum recorded rotation
φ_{pl}	Elastic component of the mid-span rotation when M_{pl} is first attained
φ_{rot}	Mid-span rotation when mid-span moment falls below M_{pl} on the descending branch
φ_u	Mid-span rotation at M_u
χ	Flexural buckling reduction factor
Ω	Upper limit for the CSM base curve
ω_g	Global geometric imperfection amplitudes

Note that (i) suffix ‘Rd’ denotes design resistance, (ii) suffixes ‘y’ and ‘z’ denote bending or buckling about major and minor axis, respectively, (iii) suffixes ‘test’ and ‘FE’ denote results obtained from experiments and FE models, respectively, (iv) suffixes ‘EC3’ and ‘AISC’ denote resistance predicted according to EN 1993-1-1 (2005, 2019) and AISC 360-16 (2016), respectively; (v) suffixes ‘mod-EC3’ and ‘prop’ denote resistance predictions determined using the new design methods proposed in the present study.

Chapter 1

Introduction

1.1 Background

In recent decades, high strength steels (HSS), generally defined as those with nominal yield strengths greater than or equal to 460 MPa, are gaining increasing use in the structural engineering industry (Bjorhovde, 2004; Graham, 2006; Miki et al., 2002; Shi et al., 2012, 2016), particularly for heavily loaded structural components in long-span bridges, high-rise buildings and stadia. Examples include the Shanghai World Financial Centre (China), the Sony Centre (Germany) (see Figure 1.1), the Yokohama Landmark Tower (Japan) and the Tseung Kwan O Cross-Bay Link Bridge (China) (see Figure 1.2). Compared with normal strength steels (NSS), the use of HSS enables the selection of smaller cross-section sizes for structural components, resulting in reductions in weight, as well as enabling more streamlined and elegant structures. In addition to material savings, a lighter structure generally requires smaller foundations and reduced transportation and construction times and costs, thereby contributing to lower CO₂ emissions and energy use.

I-sections, fabricated either by hot-rolled or by welding, are commonly used for members that are primarily subjected to bending in practical design due to their high efficiency in resisting bending moments. Welded I-sections enable the use of different steel grades for the flanges and the web, referred to as hybrid welded I-sections. Hybrid HSS welded I-sections, in which the flange plates are made of a higher strength steel than the web, are often more cost-effective than their homogenous HSS counterparts in resisting bending moment, since the web of a cross-section only contributes a modest amount to the total bending resistance; hence, it can be more

economical to use a less expensive lower strength steel in the web than in the flanges. Owing to external heat input of the welding process, welded I-sections have a residual stress distribution different from that of hot-rolled I-sections, with a higher normalised level of peak residual tension distributed within the welded area and the heat affected zone (Abambres & Quach, 2016; Gardner & Cruise, 2009). To date, investigations into the structural response and design of HSS welded I-section members and frames remain limited.



Figure 1.1: The Sony Centre, Berlin, Germany
(<https://www.arup.com/projects/sony-centre>)



Figure 1.2: The Tseung Kwan O Cross-Bay Link Bridge, Hong Kong, China
(<https://www.constructionplusasia.com/hk/cross-bay-link-tseung-kwan-o/>)

Owing to a lack of fundamental structural performance data and limitations in the existing design rules, the current codified design provisions for HSS structures, as set out in the European Standard EN 1993-1-12:2007 (EC3) and the American Specification AISC 360-16, are generally a simple extension of the design rules for NSS, without considering the full influence of the yield strength on the behaviour and design of steel structures made of varying steel grades. The majority of the existing design provisions are applicable to the design of steel structures made of steel grades up to S700, while steel grades above S700 are generally not covered. It is therefore important to examine the accuracy of the current design methods employed in predicting the ultimate capacities for HSS columns, beams and beam-columns made of steel grades up to S960. Also, it is stated in EN 1993-1-12:2007 that plastic design is not applicable to HSS indeterminate structures, due primarily to their low ductility. Although there are a number of normal strength steel frame tests reported in the literature, no experimental data are available on HSS frames, restricting the development of a safe and efficient design method for HSS indeterminate structures. Extension of plastic design to HSS indeterminate structures is studied in this thesis.

Current structural steel design provisions adopt the concept of cross-section classification that categorises cross-sections into discrete classes to account for local buckling, based on which stepwise design rules are developed for different classes of cross-sections and steps in resistance predictions result. To this end, a deformation-based method named the Continuous Strength Method (CSM) was developed to replace the concept of cross-section classification (Gardner et al., 2023). Extension of the CSM to HSS structures, within the EC3 design framework, is studied in this thesis. An advanced design method has been developed and applied to NSS structures (Fieber et al., 2019a, 2020; Quan et al., 2020, 2021), which is implemented by applying strain limits, obtained through the CSM, to the results of second order inelastic analyses of structures using beam finite elements. This method is extended to high strength steel welded I-section members in this thesis.

1.2 Research scope and objectives

In general, the aim of this research is to investigate more effective use of high strength steel in practice. The work is focussed in two key areas: (1) the improvement of traditional design approaches for normal and high strength steel structures with enhanced safety, efficiency and

consistency, and (2) the extension of the advanced method of design by second order inelastic analysis using beam finite elements to high strength steel (HSS) structures. More specifically, the objectives of this research include the following aspects:

- To investigate the structural behaviour of NSS and HSS welded I-section members and frames experimentally;
- To simulate the structural performance of NSS and HSS welded I-section members numerically;
- To review the accuracy of the current design methods for the design of NSS and HSS welded I-sections and identify potential areas for improvement;
- To propose necessary improvements to the current EC3 design methods by adopting the Continuous Strength Method (CSM);
- To extend the advanced design method for NSS structures of design by second order inelastic analysis using beam finite elements with CSM strain limits to HSS welded I-section members;
- To assess the accuracy of the advanced design method by comparing benchmark results against the resistance predictions provided using the advanced design method, as well as those obtained from the current traditional design methods;
- To provide recommendations for further research.

1.3 Overview of the thesis

This thesis consists of eight chapters. This chapter briefly introduces the advantages of high strength steels, the shortcomings of the current design standards for high strength steel structures and an overview of this thesis.

Chapter 2 presents a general review of the background literature that is related to this research, including the key topics on high strength steels, existing research on HSS determinate and indeterminate structures and existing design methods to be discussed in the thesis.

Chapter 3 investigates the flexural buckling behaviour and design of homogeneous and hybrid HSS welded I-section columns, considering a wide range of steel grades. Material tests and five pin-ended column major axis flexural buckling tests are performed. Numerical parametric investigations on pin-ended homogeneous and hybrid HSS welded I-section columns buckling

about the major and the minor axis are also presented. The developed finite element models incorporate a novel residual stress model established based on a large amount of existing residual stress measurements for welded I-sections made of S235 to S960 steels. The current EC3 design rules are then examined. A modified EC3 design method is devised within the EC3 design framework, resulting in improved accuracy and consistency.

Chapter 4 discusses the flexural behaviour and design of homogeneous and hybrid HSS welded I-section beams. An experimental programme comprising six three-point bending tests is described, followed by a numerical parametric study. The generated test and FE results are used to evaluate the suitability of the current Eurocode 3 cross-section slenderness limits for HSS homogeneous and hybrid welded I-sections in bending. Stricter Class 1 slenderness limits are deemed necessary for the extension of plastic design to HSS structures.

Chapter 5 studies the in-plane member stability and design of normal and high strength steel non-slender welded I-section beam-columns subjected to compression plus uniaxial bending numerically. The accuracy of the current EC3 design rules is examined. A modified EC3 design method for NSS and HSS welded I-section beam-columns is developed within the EC3 design framework, resulting in improved accuracy and consistency.

Chapter 6 describes eight physical tests on high strength steel frames. The tested frames have fixed-bases, are single storeyed, are unbraced in-plane and composed of welded S690 steel I-section members. Different combinations of horizontal and vertical loading are applied to the frames so that beam, sway, and combined collapse mechanisms can form. The potential for extending plastic design to HSS structures has been demonstrated.

Chapter 7 extends the advanced design method of design by second order inelastic analysis with strain limits to NSS and HSS welded I-section structural elements subjected to compression, bending and combined compression plus bending. A numerical parametric study is performed to evaluate the accuracy of the method; comparisons are made against the current EC3 design methods. Significant benefits of the advanced design method have been confirmed.

Finally, Chapter 8 summarises the key findings of this research study, as well as conclusions and suggestions for further work.

Chapter 2

Literature Review

2.1 Introduction

This chapter provides a general review of the background literature related to this thesis, covering key topics on high strength steels, previous research on determinate and indeterminate structures made of high strength steel, as well as existing design concepts and approaches. Further findings from the literature are introduced in more detail in the relevant chapters.

2.2 High strength steels (HSS)

High strength steels (HSS) are normally defined as steels with yield strengths equal to or greater than 460 MPa. They are produced by adjusting the chemical composition of the iron alloy or through heat treatments. Alloying ingredients include chromium, molybdenum, silicon, manganese, nickel and vanadium. Adjusting the percentage of the alloying ingredients can increase yield strengths (Starratt, 1958), but generally leads to reduced toughness and other variations in mechanical behaviour. Heat treatments, on the other hand, are able to greatly enhance the yield strength of structural steel, while maintaining good toughness and weldability, as well as similar mechanical behaviour to that of the normal strength structural steels (Günther & Raoul, 2005; Villalobos et al., 2018); heat treatment techniques include annealing, case hardening, precipitation strengthening, tempering, carburizing, normalizing and quenching. The present research study focuses on quenched and tempered (Q&T) high strength steels.

Figure 2.1 illustrates two typical stress-strain curves for normal and high strength steel used in structural applications; it can be seen that high strength steel exhibits very similar mechanical behaviour to normal strength steel, but with less pronounced strain hardening and lower ductility.

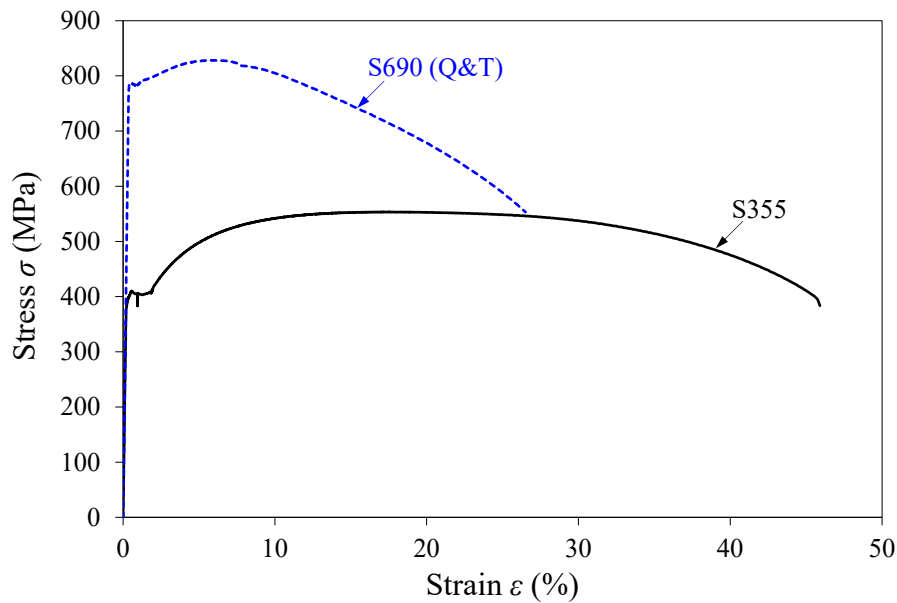


Figure 2.1: Typical measured stress-strain curves for normal strength steel (an 8-mm-thick coupon made of S355 steel) and high strength steel (an 8-mm-thick coupon made of S690 steel) obtained from the tensile coupon tests in the present study.

Recent years have seen a gradual but clear global trend towards the wider use of HSS in structural engineering, particularly in large structures with heavily loaded elements. The use of HSS can lead to significant weight savings, facilitating more streamlined, sustainable and economic structures. Sedlacek and Müller (2001) estimated that about 60% weight reduction can typically be achieved in structures when using S690 steel rather than S235 steel, which further leads to reductions in fabrication, transportation and erection costs, as illustrated in Figure 2.2. Hence, although high strength steels cost more than normal strength steels in terms of unit price, the cost penalty can be easily outweighed by the strength increase in many design cases (Baddoo & Chan, 2017). A better understanding of how to maximise the benefits of the high strength will help designers make informed decisions on material selection and related

benefits at the conceptual design of a project, thereby encouraging wider use of high strength steels in practical structural applications (Baddoo et al., 2021).

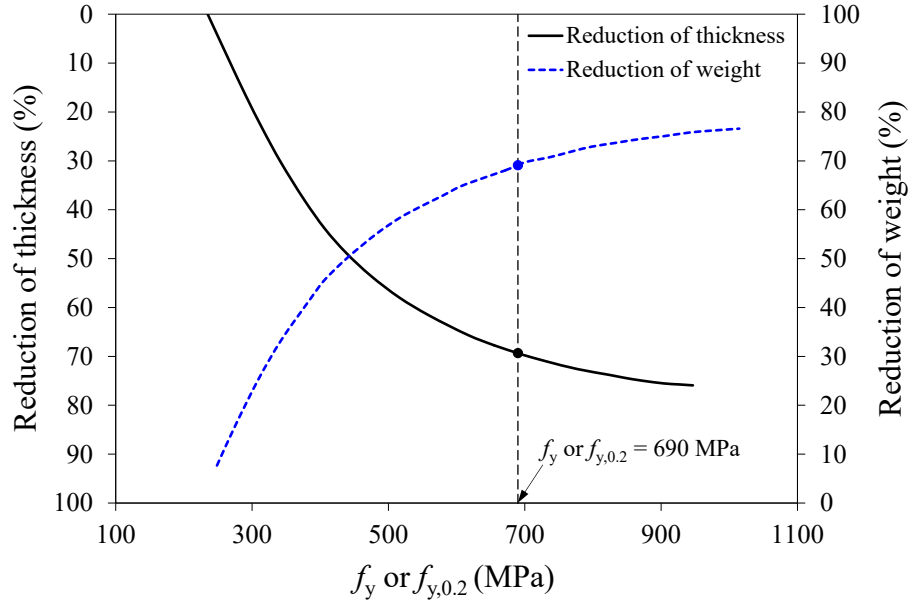


Figure 2.2: Weight and wall thickness reductions with increasing steel yield strength. (Reproduced from Sedlacek & Müller (2001) and Gogou (2012))

2.3 Previous research on HSS determinate structures

Extensive investigations have been carried out to study the structural behaviour of HSS welded I-sections over the past years through experiments and numerical simulations. The previous experiments set a benchmark for the experimental programmes performed in this research, and the collected test data are also used to examine the validity of the finite element models and to assess different design approaches. In this section, the existing test data on determinate structures (i.e. isolated structural elements) made of HSS welded I-sections under bending, compression and combined compression and bending are summarised.

2.3.1 Simply supported beams

Several three-point (moment gradient) and four-point (uniform moment) bending tests have been conducted to investigate the flexural strength and rotation capacity of HSS welded I-

sections. McDermott (1969) tested a total of nine A514 steel (i.e. with a nominal yield strength of 690 MPa) I-shaped beams subjected to either pure bending or bending with moment gradient, and highlighted the potential use of plastic design for HSS structures. Beg et al. (1996) studied the local stability of welded I-section beams made of HSS with a yield strength around 800 MPa and proposed an analytical expression to determine the Class 3 slenderness limit for HSS welded I-sections in bending considering web-flange interaction. More recently, the in-plane flexural behaviour of S690 steel welded I-section beams was investigated by Sun et al. (2019), confirming the general accuracy of the Class 2 and Class 3 slenderness limits set out in EN 1993-1-12 (2007) for S690 HSS welded I-sections in bending. The use of plastic design is not currently permitted for HSS structures in EN 1993-1-12 (2007); this restriction was questioned by Bartsch et al. (2021) who examined the rotation capacity of HSS welded I-section beams experimentally and verified that HSS beams were capable of attaining their plastic moment resistances and exhibiting good rotation capacities. A total of 69 three- and four-point bending tests on homogeneous welded I-sections made of NSS and HSS have been collected from the literature and are summarised in Table 2.1. Note that ‘3PB’ refers to three-point bending and ‘4PB’ refers to four-point bending.

Table 2.1: Summary of existing bending tests on homogeneous HSS welded I-sections.

Loading configuration	Nominal yield stress (MPa)	Number of tests	References
4PB	700	10	Beg et al. (1996)
3PB/4PB	490-800	21	Lee et al. (2013)
4PB	345	3	Shokouhian & Shi (2015)
3PB/4PB	700-960	8	Schillo & Feldmann (2018)
4PB	460-890	5	Shi et al. (2018)
3PB	550	6	Shi & Xu (2019)
4PB	690	6	Sun et al. (2019)
3PB	355-690	10	Bartsch et al. (2021)

With regards to hybrid welded I-sections, Suzuki et al. (1994) carried out a series of physical tests on simply supported beams with an HSS web and NSS flanges, observing that the plastic deformation capacity of such hybrid beams was greater than that of their homogeneous HSS

counterparts. Shokouhian and Shi (2015) conducted both experimental and numerical investigations into the flexural strength of hybrid steel I-section beams with Q460 steel flanges and a Q345 steel web subjected to uniform bending moment and proposed a slenderness-based method to determine their flexural strengths considering the effects of local buckling, global buckling and the interaction between the two. Shokouhian and Shi (2014) also studied the influence of flange and web slenderness on the ductility of beams made of HSS homogeneous and hybrid welded I-sections, revealing that stricter GB 50017 (2003) flange and web slenderness limits are required for these beams to achieve a rotation capacity of $R_{\text{req}} = 3$, which was adopted as the minimum requirement for plastic design in the development of Eurocode 3 (2005) and AISC 360 (2016). A total of 26 three- and four-point bending tests on welded I-section beams made of hybrid material (NSS web and HSS flanges) have been collected from the literature and are summarised in Table 2.2.

Table 2.2: Summary of existing bending tests on hybrid HSS welded I-sections.

Loading configuration	Nominal yield stress (MPa)	Number of tests	References
4PB	345-460	3	Shokouhian & Shi (2015)
3PB/4PB	235-485	13	Wang et al. (2016)
3PB	355-690	10	Bartsch et al. (2021)

2.3.2 Columns

A number of experimental studies have been carried out to investigate the minor axis flexural buckling behaviour of homogeneous HSS welded I-section columns, such as those reported by Ban et al. (2013), Li et al. (2016), Deng et al. (2021) and Su et al. (2021). Only a few studies have focused on buckling about the major axis, such as those reported by Shi et al. (2012) and Ban and Shi (2018). The previous investigations have shown that HSS welded I-section columns generally exhibit superior normalised flexural buckling resistances relative to their normal strength steel (NSS) counterparts, and the current stability design provisions tend to be rather conservative. A total of 25 column buckling tests on HSS welded I-sections have been collected from the literature and are reported in Table 2.3. Note that the present study only focuses on the in-plane stability of HSS welded I-sections.

Table 2.3: Summary of existing column buckling tests on homogeneous HSS welded I-sections.

Buckling axis	Nominal yield stress (MPa)	Number of tests	References
Minor axis	960	2	Ban et al. (2013)
	690	5	Li et al. (2016)
	460	8	Deng et al. (2021)
	960	10	Su et al. (2021)

2.3.3 Beam-columns

Investigations into the structural response and design of homogeneous HSS welded I-section beam-columns remain scarce. Yang et al. (2018) and Ma et al. (2017, 2018) carried out a series of tests on welded I-section beam-columns made of Q460 steel and Q690 steel, respectively, subjected to compression plus minor axis bending. They concluded that the accuracy of the existing EC3 design rules for HSS welded I-section beam-columns could be improved by introducing higher flexural buckling curves. More recently, Tse et al. (2021) performed a numerical study into the structural behaviour of S690 and S960 steel welded I-section beam-columns under compression plus uniaxial bending and proposed a modified EC3 design method for these members, providing more accurate and consistent resistance predictions than the current EC3 provisions. A total of 22 tests on HSS welded I-section beam-columns subjected to compression plus minor axis bending have been collected from the literature and are listed in Table 2.4.

Table 2.4: Summary of existing beam-column tests on homogeneous HSS welded I-sections.

Buckling axis	Nominal yield stress (MPa)	Number of tests	References
Minor axis	460	6	Li et al. (2012)
	690	8	Ma et al. (2017)
	460	8	Yang et al. (2018)

2.4 Previous research on HSS indeterminate structures

To date, very few experiments have been devoted to exploring the structural responses of indeterminate structures made of HSS. Such experimental data are much needed for the validation of numerical models and calibration of system-level design approaches. Relevant previous research is reviewed in the following subsections.

2.4.1 Continuous beams

Although investigations into the structural behaviour of continuous beams made of normal strength steel (Driscoll & Beedle, 1956; Gardner et al., 2010; Popov & Willis, 1958; Yang et al., 1952), stainless steel (Arrayago et al., 2017; Arrayago & Real, 2016; Gkantou et al., 2019; Theofanous et al., 2014; Theofanous & Gardner, 2010) and aluminium alloy (Georgantzia et al., 2022; Su et al., 2015a, 2015b, 2016) have been carried out by many researchers, experimental and numerical investigations on continuous beams made of high strength steel remain scarce. Gkantou et al. (2018) performed a series of numerical parametric studies on two-span continuous beams made of hot-finished HSS square and rectangular hollow sections and highlighted the applicability of plastic design to HSS indeterminate structures. Zhao and Ding (2022) studied the overall buckling behaviour of welded I-section two-span continuous beams made of Q460 steel through tests and a series of numerical parametric studies.

2.4.2 Frames

The cyclic behaviour and seismic performance of braced and unbraced HSS frames have been studied by Chen and Ke (2019), Hu et al. (2017) and Wang et al. (2016); however, no experimental or numerical investigation exists to explore the structural behaviour of HSS frames under static loading. This has, to a certain degree, inhibited the development of design provisions for HSS in current standards, such as the European code EN 1993-1-2 (2007) and the American Specification AISC 360 (2016), by, for example, precluding the use of plastic design. Note that scarcity of test data on structural systems is not only limited to HSS though; there is also a general dearth of high quality physical experimental results on structural steel systems. The corresponding behaviour of HSS frames is yet to be explored, and is therefore

examined experimentally in the present study, allowing the interaction between plastic collapse and global instability to be assessed.

2.5 Design approaches

In this section, key aspects of the current design approaches, an advanced design concept – the continuous strength method (CSM), and an advanced design method incorporating the CSM for structural steel design are introduced.

2.5.1 Current design approaches

Traditional structural steel design generally comprises two steps: firstly, the distributions of internal forces and bending moments are determined using a structural analysis, typically a first order elastic analysis and second order elastic analysis; secondly, a series of strength and stability checks are performed on individual structural members. The current design standards, such as the European codes EN 1993-1-1 (2005, 2019) and the American specification AISC 360 (2016), utilise member design equations to predict the ultimate capacities of the constituent structural members under the loading conditions obtained from the analysis; the effects of geometric imperfections and residual stresses are considered in the design equations. EN 1993-1-1 (2005, 2019) also includes another design approach which applies a linear interaction cross-section check to a second order elastic analysis (namely geometrically nonlinear analysis with imperfection GNIA) with equivalent member imperfections. The need for the calculation of effective lengths and separate member checks are eliminated since both the second order effects and the member instability are directly captured in the finite element models.

Traditional design approaches are based on the assignment of cross-sections into discrete behavioural classes according to their susceptibility to local buckling. In the case of bending, Class 1 cross-sections can fully reach their plastic bending moment capacity M_{pl} and have sufficient rotation capacity ($R \geq 3$) to form plastic hinges (where R is defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield prior to significant load shedding (AISC 360-16, 2016)), and are suitable for plastic design. Class 2 cross-sections can

also fully reach their plastic bending moment capacity M_{pl} but have lower rotation capacity in bending ($R < 3$), and are only suitable for elastic design. Class 3 cross-sections can fully achieve their elastic bending moment capacity M_{el} but fail to fully develop the plastic bending moment capacity M_{pl} due to local buckling. The latest version of EN 1993-1-1 (2019) introduces an elasto-plastic bending capacity M_{ep} , which is a linear interpolation between M_{pl} and M_{el} , as the bending moment capacity for Class 3 cross-section, eliminating the artificial step in bending moment resistance between Class 2 (M_{pl}) and Class 3 (M_{el}) cross-sections set out in EN 1993-1-1 (2005). Class 4 cross-sections are those in which local buckling occurs before the attainment of yield strength in any part of the cross-section, whose moment resistance are taken as the effective bending moment capacity M_{eff} determined using the effective width method given in EN 1993-1-5 (2006). The concept of cross-section classification offers a simplified and straightforward method to account for the influence of local buckling on cross-sections; however, it still has some limitations, as described below. Firstly, the concept ignores material strain hardening in the determination of the bending moment capacities for stocky cross-sections, conservatively limiting the cross-sectional bending moment resistance to the plastic bending moment capacity M_{pl} (Gardner et al., 2011). Secondly, the cross-section classification framework disregards the beneficial effect of local moment gradients on bending resistances, leading to conservative resistance predictions for members subjected to moment gradients; this has been widely recognised by many researchers (Chan & Gardner, 2008; Chen & Young, 2019b, 2019a; Wang et al., 2016c).

The European design standard for HSS structures EN 1993-1-12 (2007) adopts most of the design rules in EN 1993-1-1 (2005), but states that clauses 5.4.1(3) and 5.4.1(4)B of EN 1993-1-1 (2005) are not applicable to HSS; this effectively means that plastic redistribution of forces and moments and plastic design are not permitted. Partial plastification of members is allowed, but the permitted degree of plasticity is not stated. The design rules developed for NSS are directly applied to HSS, without fully considering the influence of yield strength. The prohibition of plastic design for HSS also makes Class 1 cross-sections not necessary to be classified. Since the potential of applying plastic design to HSS indeterminate structures has been demonstrated in some recent research (Bartsch et al., 2021b; Gkantou et al., 2018), it is needed to reassess the current design rules for HSS and to extend plastic design to HSS structures, which are the focuses of the present study.

2.5.2 Advanced design methods

To overcome the above-mentioned limitations of the cross-section classification framework, an advanced design concept named the continuous strength method (CSM) was proposed and developed over the past two decades (Gardner, 2008; Gardner et al., 2023). The CSM is a deformation-based method that replaces the concept of cross-section classification with a continuous relationship (i.e. the CSM base curve) between cross-section slenderness $\bar{\lambda}_p$ and deformation capacity (strain ε). The CSM base curve, given by Equation (2.1) for plated cross-sections, defines the limiting strain ε_{csm} that a cross-section can endure prior to reaching its ultimate capacity. In the equation, cross-section slenderness $\bar{\lambda}_p$ is equal to square root of the yield strength f_y divided by the elastic local buckling stress of the cross-section $\sigma_{\text{cr,cs}}$, as given by Equation (2.2), ε_y is the yield strain equal to the yield strength f_y divided by the Young's modulus E , ε_u is the ultimate strain estimated as $\varepsilon_u = 0.6(1-f_u/f_y)$ yet no less than 0.06 for hot-rolled steels, in which f_u is the ultimate strength and C_1 is a coefficient corresponding to a quad-linear material model developed by Yun and Gardner (2017). Two upper bounds are applied to the strain ratio $\varepsilon_{\text{csm}}/\varepsilon_y$: the first limit of Ω is set to prevent excessive strains and defines the permissible level of plastic deformation on a project-by-project basis; the second limit of $C_1\varepsilon_u/\varepsilon_y$ defines a cut-off strain to avoid over-predictions of material strength when using the CSM resistance functions developed for hand calculations (Gardner et al., 2023; Yun et al., 2018a; Yun & Gardner, 2017). In this research study, values of $\Omega = 15$ (generally recommended in line with the EN 1993-1-1 (2005) ductility requirement) and $\Omega = 10$ are considered.

$$\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} = \begin{cases} \frac{0.25}{\bar{\lambda}_p^{3.6}} \leq \min\left(\Omega, \frac{C_1\varepsilon_u}{\varepsilon_y}\right) & \text{for } \bar{\lambda}_p \leq 0.68 \\ \left(1 - \frac{0.222}{\bar{\lambda}_p^{1.05}}\right) \frac{1}{\bar{\lambda}_p^{1.05}} & \text{for } \bar{\lambda}_p > 0.68 \end{cases} \quad (2.1)$$

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{\text{cr,cs}}}} \quad (2.2)$$

Design by advanced analysis refers to any design method that captures the strength and stability of a structural system and its individual members in such a way that separate member capacity

checks are not required; for frame design, these analyses are also known as second order inelastic analysis (Chen, 2008). Compared with the traditional two-step design process, as described in Section 2.5.1, advanced analysis offers a more straightforward design process that considers both the geometric and material nonlinearities and leads to a more realistic and accurate representation of the actual behaviour of steel structures. Advanced analysis is commonly carried out using beam finite elements with geometric imperfections and residual stresses being suitably considered. Owing to improvements in computational power and software, system-level advanced analyses are increasingly used in engineering practice and have been extensively explored over the past decades (Buonopane & Schafer, 2006; Chan & Chen, 2005; Gardner et al., 2019; Zhang et al., 2016; Ziemian, 1990; Ziemian et al., 1992a, 1992b). Recently, a practical advanced analysis-based design method that incorporates the CSM has been developed for structural steel design (Fieber et al., 2019a, 2020; Gardner et al., 2019; Murtaza & Kucukler, 2022; Quan et al., 2020, 2021; Walport et al., 2021, 2023). This advanced design method is implemented by (1) performing a geometrically and materially nonlinear analysis with imperfections (GMNIA), i.e. second order inelastic analysis, on a beam finite element model, and (2) applying strain limits obtained from the continuous strength method (CSM) to determine the ultimate capacity of the structure. The CSM strain limit is employed to allow for the effects of local buckling, and hence control the spread of plasticity through the cross-section and within the structure to a level that is related to the local slenderness of the constituent cross-sections, thereby discarding the need for cross-section classification. This design approach has been developed and applied to structures made of normal strength steel (Fieber et al., 2019a, 2020; Gardner et al., 2019; Murtaza & Kucukler, 2022; Quan et al., 2020, 2021) and stainless steel (Walport et al., 2021, 2023), providing significant benefits over the current design approaches. This advanced design method is extended to HSS structures in the present research study.

2.6 Concluding remarks

This chapter provided an overview of the key topics relevant to this thesis. More detailed analysis of the relevant literature was presented. The history and advantages of high strength steels were firstly introduced. A summary of previous research on the structural performance

of determinate and indeterminate structures made of HSS was then presented, which highlighted the lack of high quality physical experimental results on HSS, particularly for indeterminate structures. Finally, the key aspects of the current design approaches, an advanced design concept – the continuous strength method (CSM), and a practical advanced analysis design method were described. The need for reassessment and improvement to the current design approaches for high strength steel, and extension of the advanced CSM-based design concept to HSS structures were demonstrated, which are the focus of this thesis, henceforth.

Chapter 3

Welded I-section columns

3.1 Introduction

Chapter 3 aims to characterise the flexural buckling behaviour and resistance of both homogeneous and hybrid welded I-section columns with varying steel grades through both experimental and numerical studies. Measured residual stress data from the literature were firstly collected and analysed to develop a novel residual stress model. An experimental programme was then conducted to generate flexural buckling data on homogeneous and hybrid welded I-section columns buckling about the major axis; a comprehensive numerical investigation to generate further column flexural buckling resistance data over a broader variety of steel grades, cross-section geometries and member slendernesses followed. The obtained test and numerical data are utilised to examine the accuracy and applicability of the codified column design rules, as specified in prEN 1993-1-1 (2019) and AISC 360 (2016), for both homogeneous and hybrid welded I-section columns. Finally, new design proposals are made to improve the accuracy and consistency of the flexural buckling resistance predictions; the reliability of the new design approach was verified by means of reliability analyses.

3.2 Residual stress model

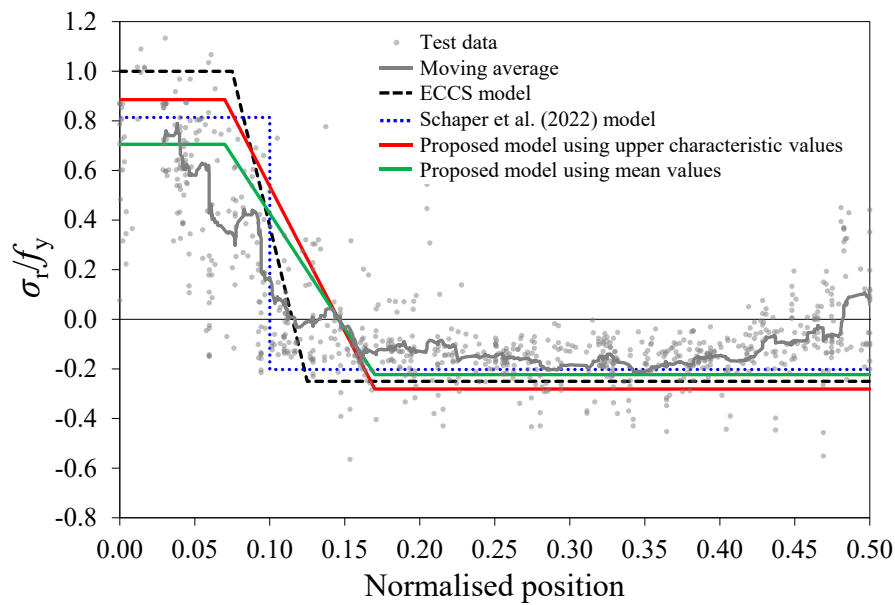
Residual stresses are introduced into welded I-sections due primarily to the non-uniform cooling that takes place during and after the welding process. The flexural buckling resistances

of welded I-section columns can be strongly affected by residual stresses. Measurements of residual stresses in both NSS and HSS welded I-sections have been carried out in a number of previous studies (Le et al., 2019; Li et al., 2015; Liu & Chung, 2018; Wang et al., 2012), resulting in the proposal of different residual stress predictive models with varying complexity. It has been shown that the amplitudes of residual stresses in welded I-sections decrease as a proportion of the yield strength with increasing steel grade. A review of residual stress measurements and models for welded steel I-sections has been made by Tankova et al. (2019); on the basis of the collected residual stress measurements, statistical values of the amplitudes of residual stresses in welded I-sections with varying steel grades have also been provided. More recently, Schaper et al. (2022) performed residual stress measurements on mono- and doubly-symmetric welded I-sections with steel grades ranging from S355 to S690, and proposed a new residual stress model that considers both cross-section geometry and steel grade. The residual stress amplitudes employed in the new model (Schaper et al., 2022) correspond approximately to the mean values of the residual stress amplitudes in compression. An alternative approach is taken in the present study – a residual stress model with two alternative residual stress amplitudes is proposed in this section. Existing residual stress measurement data on doubly symmetric welded I-sections made of steel plates with nominal yield strengths ranging from 235 MPa to 960 MPa are analysed; the data collected from the literature are summarised in Table 3.1.

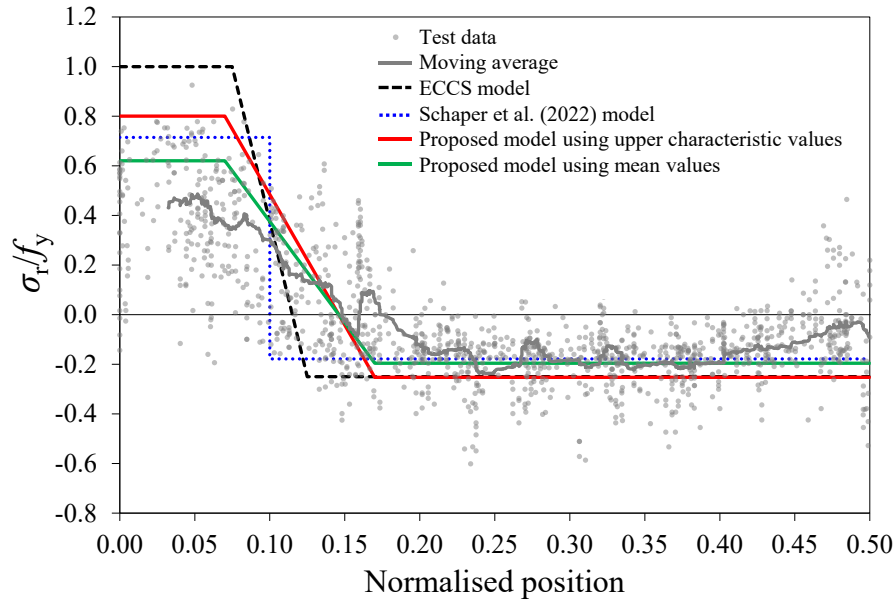
The collected measured residual stress data for doubly symmetric welded I-sections made of S355, S460 and S690 steels are presented in a normalised fashion in Figure 3.1, where the measured residual stresses σ_r are normalised by their corresponding yield strengths f_y and plotted against the normalised positions with respect to the flange width or web depth of the measured welded I-section. Note that in Figure 3.1, the origin of the horizontal axis corresponds to the web-to-flange junction while the value of 0.5 represents the location of the flange tip or the mid-height of the web of the measured welded I-section. Also note that throughout the present study positive values indicate tensile residual stresses while the negative values indicate compressive residual stresses. Moving averages over 40 adjacent data points of measured residual stresses for each considered steel grade are also plotted in Figure 3.1, together with the ECCS model (ECCS, 1984; prEN 1993-1-14, 2019) and the predictive model for welded I-sections made of non-thermally cut plates recently proposed by Schaper et al. (2022).

Table 3.1: Existing residual stress measurements on doubly symmetric welded I-sections

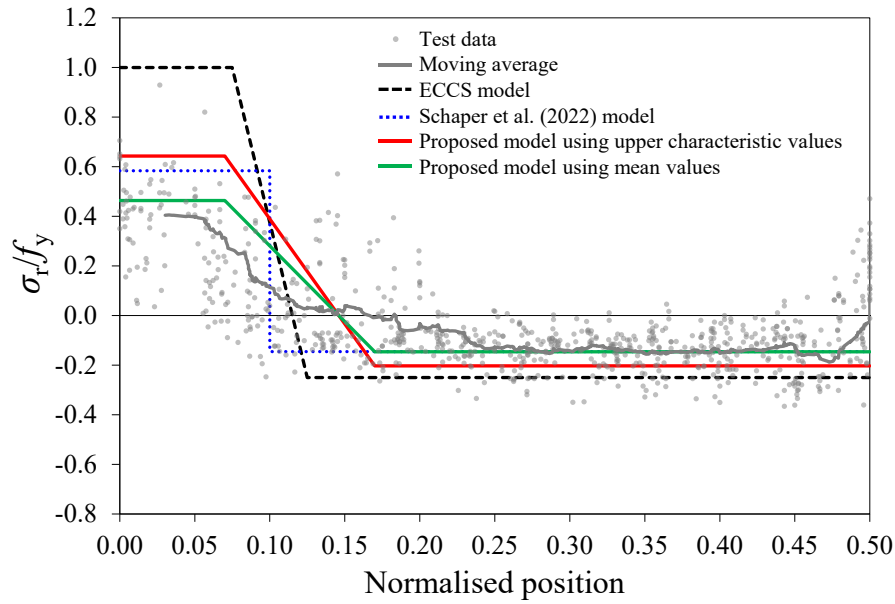
Nominal yield strength (MPa)	No. of tests	Member type	Reference
235	1	Prismatic	Wang and Qin (2016)
	3	Tapered	Shiomi and Kurata (1984)
345/355/350	4	Tapered	Tankova et al. (2019)
	4	Prismatic	Unsworth et al. (2020)
	4	Prismatic	Yang et al. (2017)
	7	Prismatic	Schaper et al. (2019)
460	8	Prismatic	Ban et al. (2013)
	8	Prismatic	Yang et al. (2016)
	2	Prismatic	Schaper et al. (2019)
	2	Prismatic	Tankova et al. (2021)
	2	Prismatic	Zhao and Ding (2022)
690	2	Prismatic	Sun et al. (2019)
	1	Prismatic	Le et al. (2019)
	4	Prismatic	Liu and Chung (2018)
	3	Prismatic	Li et al. (2015)
	1	Prismatic	Su et al. (2020)
	4	Prismatic	Tankova et al. (2021)
	5	Prismatic	Schaper et al. (2022)
890	1	Prismatic	Le et al. (2019)
960	3	Prismatic	Ban (2012)
S690 (flange) + S355 (web)	2	Prismatic	Schaper et al. (2022)



(a) S355 steel welded I-sections



(b) S460 steel welded I-sections



(c) S690 steel welded I-sections

Figure 3.1: Comparisons between measured residual stress data and different predictive models

The general residual stress pattern adopted in the two existing models, i.e. the ECCS and the Schaper et al. (2022) models, is illustrated in Figure 3.2, with the key parameters listed in Table 3.2, where $\sigma_{r,wt}$ and $\sigma_{r,ft}$ are the maximum tensile residual stresses in the web and the flanges, respectively, $\sigma_{r,wc}$ and $\sigma_{r,fc}$ are the maximum compressive residual stresses in the web and the flanges, respectively, and a , b , c and d are stress distribution parameters. It should be noted that

the predictive model proposed by Schaper et al. (2022) does not feature a linear transition from the compressive to tensile residual stress regions (i.e. the distribution parameters a and d are equal to 0), leading to a stepwise residual stress distribution. It can be seen from Figure 3.1 that the ECCS model generally overestimates the residual stress amplitudes and fails to capture the transitional region from tension to compression accurately, particularly for welded I-sections made of the higher steel grades. The residual stress model proposed by Schaper et al. (2022) is shown to be capable of accurately capturing the trend of the reducing relative residual stress amplitudes with increasing steel grades; the model does not however represent the presence of the transition regions.

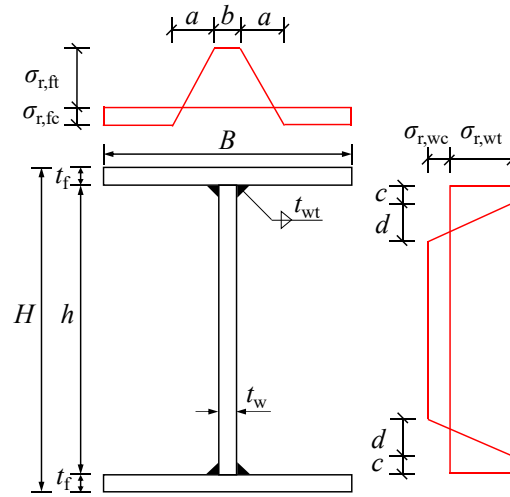


Figure 3.2: Proposed residual stress model

Table 3.2: Parameters employed in different residual stress models

	$\sigma_{r,wt} = \sigma_{r,ft}$	$\sigma_{r,wc} = \sigma_{r,fc}$	a	b	c	d
ECCS (1984, 2019)	f_y	$-0.25f_y$	$0.05B$	$0.15B$	$0.075h$	$0.05h$
Schaper et al. (2022)	$f_y \varepsilon^*$	From equilibrium	0	$\min\{t_w + 5t_{wt}^{**}, B/5\}$	$0.1h$	0
Proposed model	Equation (3.1) or (3.2)	From equilibrium	$0.1B$	$0.14B$	$0.07h$	$0.1h$

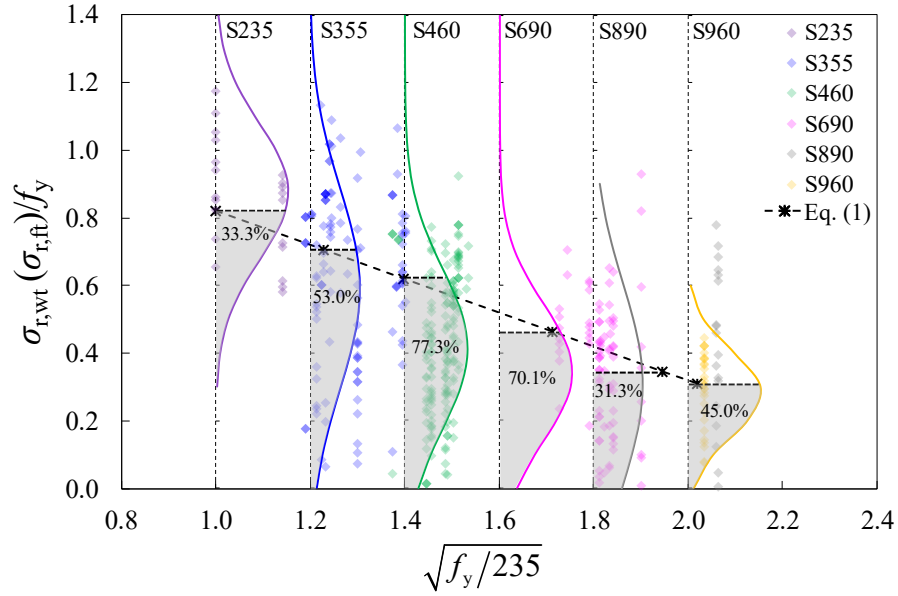
* $\varepsilon = (235/f_y)^{0.5}$, where f_y is the yield strength of the corresponding plate; ** t_{wt} is the weld throat thickness, as defined in Fig. 2.

Based on the comprehensive assembled collection of experimental data, believed to be the largest gathered to date, a new residual stress model is proposed herein. The proposed residual stress model adopts the general pattern of the conventional ECCS model, but the key

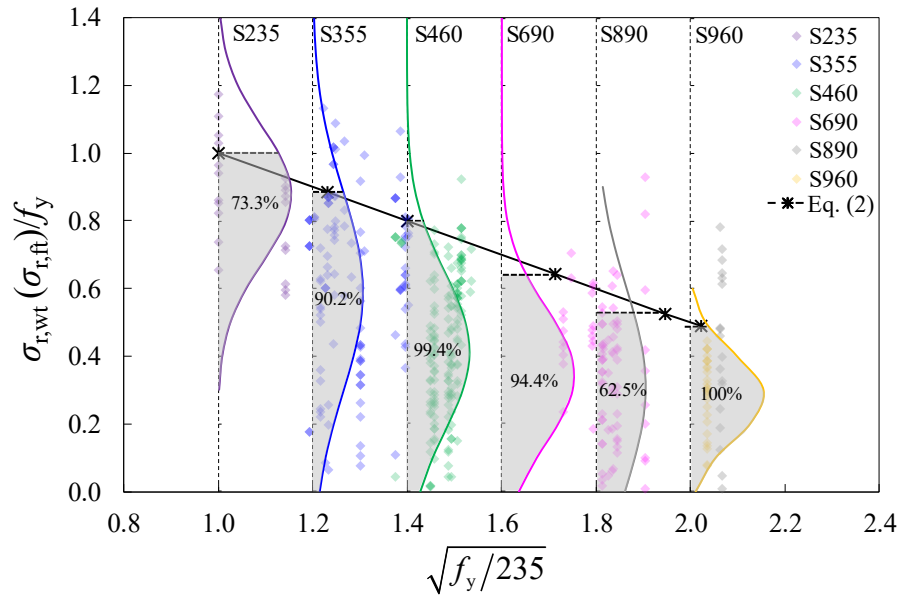
parameters have been re-calibrated against the extensive experimental data, as summarised in Table 3.2. Compared to the ECCS model, the width of the peak tensile residual regions (i.e. the parameters b and c for the flanges and web respectively) in the proposed model has been marginally reduced, while the width of the linear transition regions (i.e. the parameters a and b for the flanges and web respectively) has been extended from $0.05B$ to $0.1B$. The maximum tensile residual stresses ($\sigma_{r,wt}$ and $\sigma_{r,ft}$) can be determined from Equation (3.1) or Eq. (3.2), which correspond approximately to the mean or upper characteristic (i.e. 95 percentile) values from the analysed experimental database, as indicated in Figures 3.3 (a) and (b), respectively. It should be noted that for each of the investigated steel grades, the measured maximum tensile residual stress data were found to follow an approximately normal distribution, with all statistical information obtained by means of the probabilistic modelling approach outlined in (Arrayago et al., 2020; Shayan et al., 2014). Also note that tensile residual stresses exist at the flange tips of welded I-sections fabricated from flame-cut steel plates, but these were, conservatively, not considered in the present proposals, to ensure the general applicability of the model. The maximum compressive residual stresses ($\sigma_{r,wc}$ and $\sigma_{r,fc}$) can then be derived based on self-equilibrium upon determination of the maximum tensile residual stresses. The proposed residual stress models with the mean and upper characteristic maximum tensile residual stresses (determined by Equations (3.1) and (3.2), respectively) are also plotted in Figure 3.1, which are shown to generally agree better with the distributions of the experimental data in comparison with the existing models.

$$\frac{\sigma_{r,wt}(\sigma_{r,ft})}{f_y} = -0.5 \times \sqrt{\frac{f_y}{235}} + 1.32 \leq 1 \quad (3.1)$$

$$\frac{\sigma_{r,wt}(\sigma_{r,ft})}{f_y} = -0.5 \times \sqrt{\frac{f_y}{235}} + 1.5 \leq 1 \quad (3.2)$$



(a) Statistical evaluation of Equation (3.1)



(b) Statistical evaluation of Equation (3.2)

Figure 3.3: Comparisons between the measured residual stresses within peak tensile residual stress regions and the (a) mean and (b) upper characteristic (95 percentile) maximum tensile residual stresses predicted by Equations (3.1) and (3.2) respectively, together with statistical distribution diagrams for each of the investigated steel grades.

For doubly symmetric hybrid welded I-sections, the maximum tensile residual stresses in the web $\sigma_{r,wt}$ and the flanges $\sigma_{r,ft}$ should also be determined using Equation (3.1) or (3.2), but with the value of f_y taken as the greater yield strength of the constituent plates. In cases where the predicted value of the maximum tensile residual stress exceeds the yield strength of the

corresponding plate, the maximum tensile residual stress should be taken equal to the yield strength of that plate. As noted in the introduction, it is recommended that Equation (3.1) is used in the validation of FE models, while Equation (3.2) is used for FE parametric studies and design by advanced analysis (Fieber et al., 2019a, 2020; Gardner et al., 2019). This is consistent with the approach typically taken for the treatment of initial geometric imperfections.

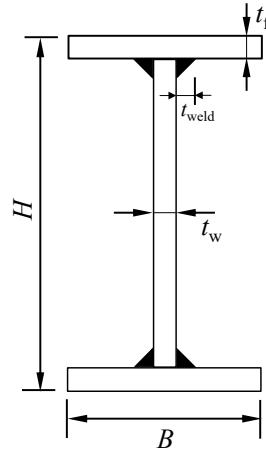
3.3 Experimental investigation

3.3.1 General

An experimental programme was carried out to address the sparsity of test data on the major axis flexural buckling behaviour of HSS homogeneous and hybrid welded I-section columns. Three welded I-section profiles were investigated in the experimental programme: two homogeneous S690 welded I-sections – I-65×116×8×8 (flange width $B \times$ height $H \times$ flange thickness $t_f \times$ web thickness t_w in mm, labelled “HSS-I1”) and I-80×136×8×8 (labelled “HSS-I2”), and a hybrid welded I-section I-80×136×8×8 (labelled “HYB-I3”). The two homogeneous S690 welded I-sections were fabricated from 8 mm-thick quenched and tempered (QT) S690 steel plates, while the hybrid welded I-section comprised flanges fabricated from QT S690 steel plates and a web fabricated from an 8 mm-thick hot-rolled S355 steel plate; the chemical composition of the employed S690 and S355 steels, as listed in the mill certificates, are provided in Table 3.3. All three investigated welded I-sections were fabricated by means of gas metal arc welding (GMAW) to produce fillet welds with a nominal weld leg length t_{weld} of 5.6 mm. The geometrical configuration and adopted notation for the welded I-sections are shown in Figure 3.4. A total of five welded I-section column specimens were fabricated, including one for each of the three investigated profiles and two repeated test specimens. The identifier for each column specimen (e.g. HSS-I1-C-R) consisted of the cross-section label (i.e. HSS-I1, HSS-I2 or HYB-I3), the letter “C” to denote a column and the letter “R” to denote a repeat test. The experimental programme included tensile coupon tests, initial global geometric imperfection measurements and major axis flexural buckling tests, as detailed in the following subsections.

Table 3.3: Chemical composition for 8 mm-thick S355 and S690 parent plates

Steel grade	C %	Si %	Mn %	P %	S %	N %	Cu %	Mo %	Ni %	Cr %	V %	Nb %	Ti %	B %	Zr %	Al %
S355	0.165	0.475	1.548	0.013	0.0008	-	0.023	0.01	0.029	0.031	0.001	-	-	-	-	0.042
S690	0.139	0.284	1.436	0.013	0.0007	0.0029	0.026	0.068	0.029	0.323	0.002	0.022	0.011	0.0016	0.0002	0.031

**Figure 3.4:** Geometrical configuration and adopted notation for welded I-sections

3.3.2 Tensile coupon tests

Tensile coupon tests were carried out to acquire the key material properties and full-range stress-strain curves of the studied S690 and S355 steels. Details regarding the dimensions of the tensile coupons as well as the coupon test setup and loading procedures are shown in Figure 3.5, while the key results are briefly summarised in this subsection. For the two investigated steel grades, two tensile coupons were extracted from each 8 mm-thick parent steel plate along the rolling direction which coincided with the longitudinal direction of the column specimens. The dimensions of the tensile coupons (in mm) are shown in Figure 3.6(a), while the measured engineering stress-strain curves for the two examined steel grades are shown in Figure 3.6(b). The average measured values of the key material properties, including the Young's modulus E , the yield strength f_y , the ultimate strength f_u , the strain hardening strain ε_{sh} where the yield plateau ends and subsequently the strain hardening initiates (Yun & Gardner, 2017), the strain at the ultimate strength ε_u and the fracture strain ε_f measured over a gauge length of 140 mm, are summarised in Figure 3.4. The ultimate-to-yield strength ratio f_u/f_y and strain ratio $\varepsilon_u/\varepsilon_y$ (where $\varepsilon_y = f_y/E$) are also given in Figure 3.4, which are shown to satisfy the material ductility

requirements specified in EN 1993-1-1 (2005) and EN 1993-1-12 (2007) for NSS and HSS, respectively.

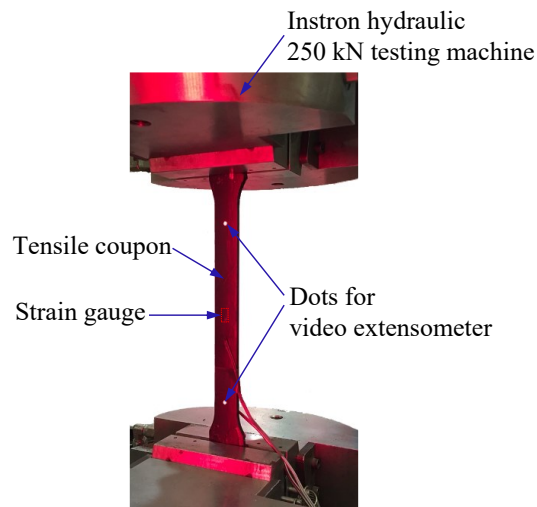


Figure 3.5: Test setup for tensile coupons.

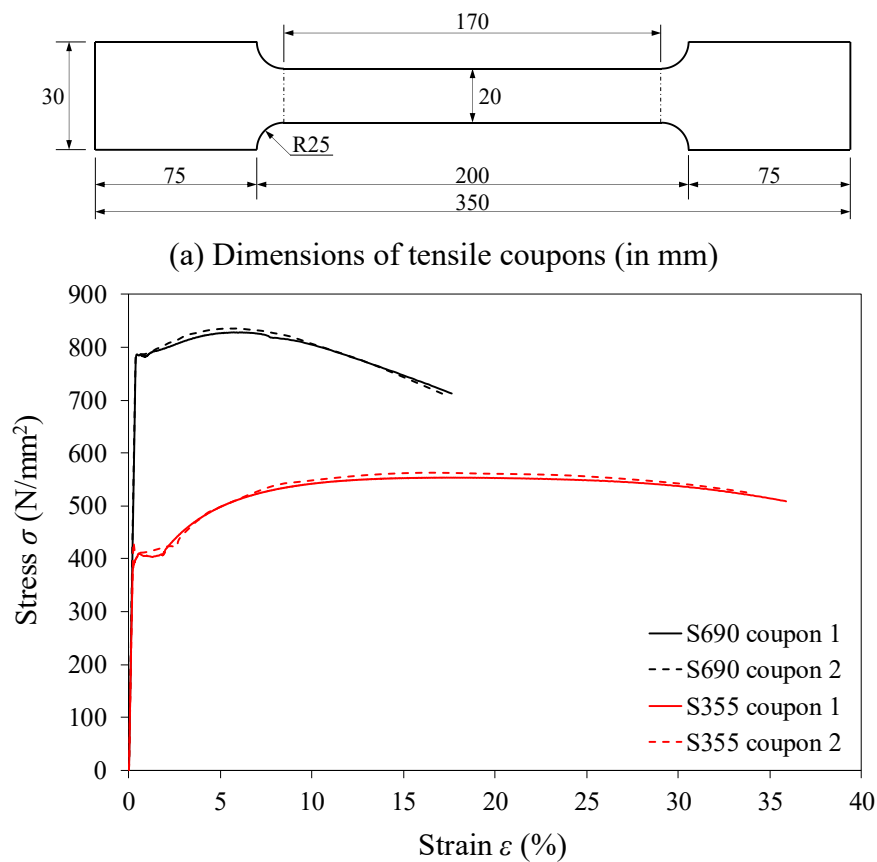


Figure 3.6: Tensile coupon tests conducted in the present study.

Table 3.4: Average measured material properties for S355 and S690 steels from tensile coupon tests

Steel grade	E N/mm ²	f_y N/mm ²	f_u N/mm ²	ε_y %	ε_{sh} %	ε_u %	ε_f %	f_u/f_y -	$\varepsilon_u/\varepsilon_y$ -
S355	198500	404.1	553.5	0.20	1.89	17.22	34.8	1.37	84.6
S690	212000	782.5	828.4	0.37	0.96	6.17	19.3	1.06	16.7

3.3.3 Imperfection measurements

Measurements were taken of initial major axis global geometric imperfections (i.e. out-of-straightness in the direction of buckling) for all five column specimens prior to testing. The test setup for the measurement of the initial global geometric imperfections is shown in Figure 3.7, in which the column specimens were clamped to a horizontal bed and two pairs of LVDTs were secured to a trolley that could move up and down a straight aluminium track along the length of the specimens. The same test setup was also successfully employed in (Behzadi-Sofiani et al., 2021; Kyprianou et al., 2021). The LVDTs were utilised to record the variation in the out-of-straightness along the flange tips over the full column length. The amplitude of the initial global geometric imperfections w_g was taken as the maximum value of the deviations of the recorded displacement readings from a reference datum that passes through the displacement readings recorded at the member ends, as illustrated by the blue dashed line in Figure 3.7. The measured imperfection amplitudes w_g are reported in Table 3.5; all were less than the value of $L_{cr}/1000$, where L_{cr} is the critical effective buckling length of the column specimens between the knife edges.

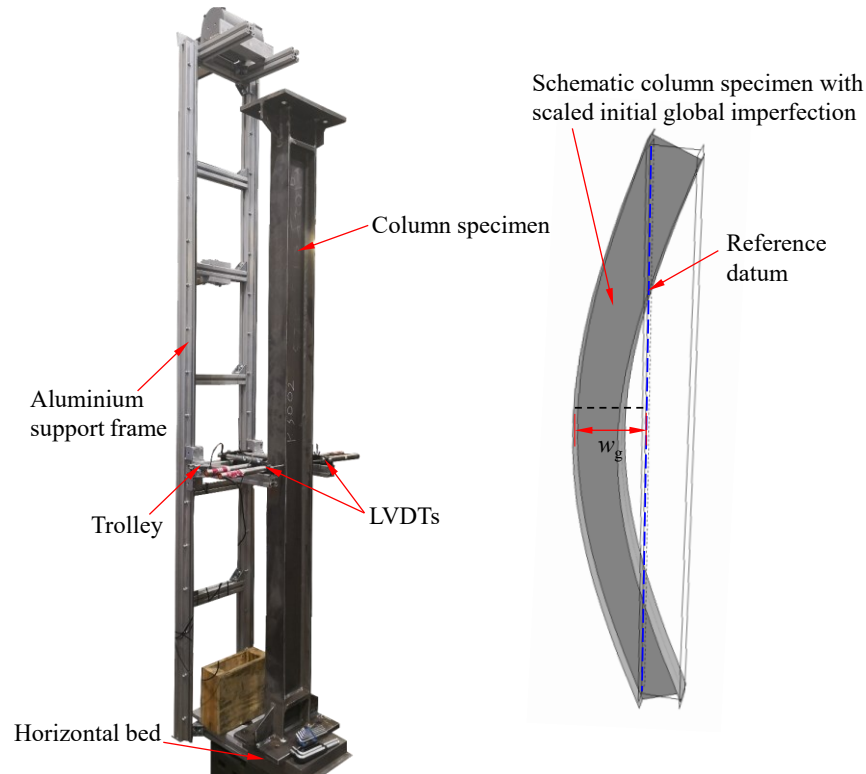


Figure 3.7: Test setup for measurement of initial global geometric imperfections and definition of w_g

Table 3.5: Average measured geometric properties and key results from column buckling tests

Specimen	B mm	H mm	t_f mm	t_w mm	t_{weld} mm	L mm	L_{cr} mm	A mm ²	$\bar{\lambda}$ -	w_g mm	e_g mm	N_u kN	δ_u m
HSS-I1-C	65.04	116.77	8.36	8.42	7.08	2000	2182	2028.6	0.94	0.85	2.38	1158.2	6.9
HSS-I1-C-R	65.12	116.60	8.36	8.33	6.71	2000	2182	2010.4	0.94	0.77	2.21	1079.2	5.3
HSS-I2-C	79.58	137.24	8.36	8.26	5.10	2000	2182	2378.3	0.78	1.10	2.40	1418.1	6.0
HYB-I3-C	79.97	136.06	8.32	8.16	7.09	2000	2182	2407.9	0.71	1.08	2.19	1242.5	6.8
HYB-I3-C-R	79.87	136.68	8.37	8.17	6.15	2000	2182	2394.8	0.70	0.92	2.26	1243.5	5.8

3.3.4 Major axis flexural buckling tests

A total of five pin-ended major axis flexural buckling tests on homogeneous and hybrid welded I-section columns were conducted. The average measured geometric properties for the five column specimens are reported in Table 3.5. The nominal length L of all column specimens was 2000 mm and two 16 mm-thick end plates were welded onto both ends of the specimens. Stiffeners were also welded at each end of the column specimens to replicate the conditions in

parallel tests on structural frames (see Chapter 6), though the stiffeners would be expected to have negligible influence on the column buckling behaviour, as shown in Figure 3.8.

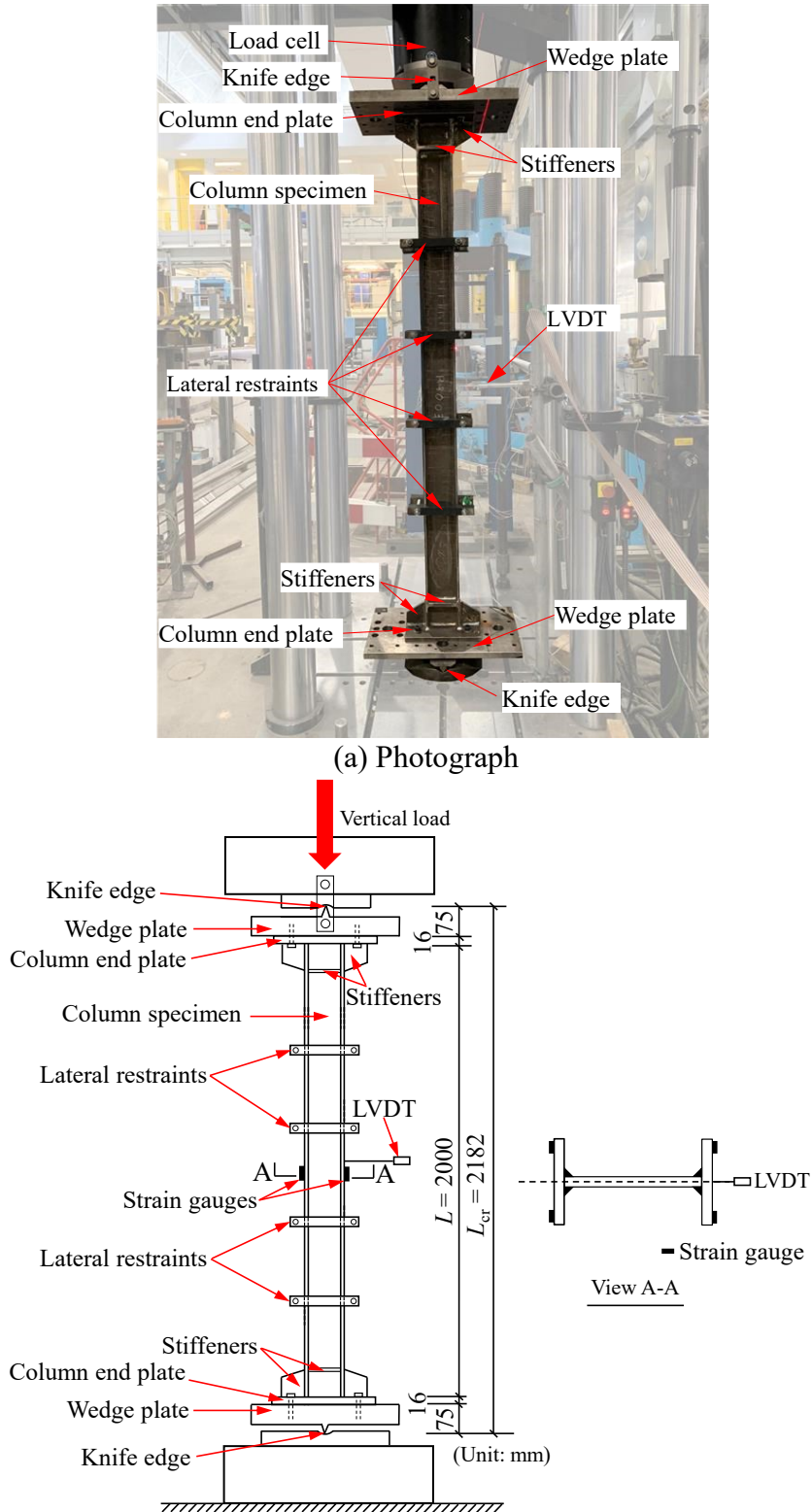


Figure 3.8: Test setup for pin-ended major axis flexural buckling tests

The column specimens were bolted to the top and bottom wedge plates, and the pin-ended boundary conditions were achieved by means of a pair of knife edges located at a distance of 75 mm from the corresponding column end plate, as illustrated in Figure 3.8. The knife edges allowed the column specimens to rotate freely around their major axis but rotation was fixed around the minor axis, leading to an critical effective length L_{cr} for major axis flexural buckling equal to 2182 mm (i.e. the sum of the column specimen length, two column end plate thicknesses and two distances between the knife edge and the corresponding column end plate) for each column specimen. The critical effective length L_{cr} was used in the determination of the non-dimensional member slenderness $\bar{\lambda}$, as defined by Equation (3.3) for members with non-slender cross-sections (as was the case for all specimens tested herein), where $N_{c,R}$ is the cross-sectional compression resistance and N_{cr} is the elastic (Euler) buckling load.

$$\bar{\lambda} = \sqrt{\frac{N_{c,R}}{N_{cr}}} = \sqrt{\frac{Af_y}{N_{cr}}} \quad (3.3)$$

For the hybrid welded I-section columns, the weighted average yield strength $f_{y,a}$ was employed for determining $N_{c,R}$ throughout the present study, as given by Equation (3.4), where $f_{y,f}$ and $f_{y,w}$ are the yield strengths of the flange plates and the web plate, respectively, A_f and A_w are the total areas of the flange plates and the web plate, respectively, and A is the cross-sectional area. The non-dimensional member slenderness $\bar{\lambda}$ for each column specimen is provided in Table 3.5.

$$f_{y,a} = \frac{f_{y,f}A_f + f_{y,w}A_w}{A} \quad (3.4)$$

All column flexural buckling tests were carried out using an Instron 2000 kN capacity testing machine. For each of the column specimens, four strain gauges were mounted at positions adjacent to the flange tips (at a distance of approximately 5 mm from the tip edges) at mid-height, with a pair of strain gauges on each flange face as illustrated in Figure 3.8, to measure the outer fibre strain histories at the critical section. A linear variable displacement transducer (LVDT) was attached at the mid-height of each column specimen, as shown in Figure 3.8, for the measurement of the mid-height lateral deflections. The initial loading eccentricity e_0 of each

column test could be slightly adjusted within the 2 mm clearance between the bolt shanks and the holes in the wedge plates. For each column test, e_0 was adjusted such that the amplitude of the total column global imperfection e_g , equal to the measured initial global geometric imperfections w_g plus the initial loading eccentricity e_0 , reached a value as close as possible to $L_{cr}/1000$, which is conventionally assumed in the development of column buckling design curves (Meng & Gardner, 2020a; Yun et al., 2022; Ziemian, 2010). The adjustment procedure was performed as follows: (1) prior to the actual testing, each column specimen was preloaded to approximately 20% of its expected failure load and the preload outputs, including the applied load and the strain gauge and LVDT readings, were recorded; (2) the present column global imperfection e_g was then determined using Equation (3.5), where I is the second moment of area of the welded I-section about its major axis, $\varepsilon_{\max}$ and $\varepsilon_{\min}$ are the average strains obtained from the strain gauges on the concave and convex flanges of each column specimen (see Figure 3.8), respectively, and N and Δ are the applied axial load and the corresponding mid-height lateral deflection measured by the LVDT, respectively; (3) the column position was then carefully adjusted and the above process repeated until the total column global imperfection e_g reached a value sufficiently close to $L_{cr}/1000$. The same approach to the positioning of column test specimens has been successfully employed in (Meng & Gardner, 2020a; Zhong et al., 2022). The final values of e_g for all column specimens are provided in Table 3.5.

$$e_g = \frac{EI(\varepsilon_{\max} - \varepsilon_{\min})}{HN} - \Delta \quad (3.5)$$

A bespoke restraint system was designed to prevent minor axis buckling of the column specimens without inducing any undesirable influence on their major axis flexural buckling behaviour. The restraint system was composed of a reaction frame and four lateral restraints, as shown in Figure 3.9. Each lateral restraint comprised two tee-shaped plates that were placed on either side of the web of the column specimens to clamp the corresponding section by means of two M30 bolts, and a bracing member comprising two parallel square hollow section (SHS) tubes made of S460 steel, as shown in Figure 3.10. The assembled tee-shaped plates were connected to the bracing member using an M24 bolt. The shank of the M24 bolt was greased so that the assembled tee-shaped plates would be able to rotate around the bolt shank. Two Macalloy bars with 35 mm diameters, as shown in Figure 3.10, were used in each lateral

restraint; one allowed adjustment of the length of the bracing member as well as the rotation of the lateral restraint about the axis of the bar, while the other permitted horizontal movement of the lateral restraint in the direction of buckling.

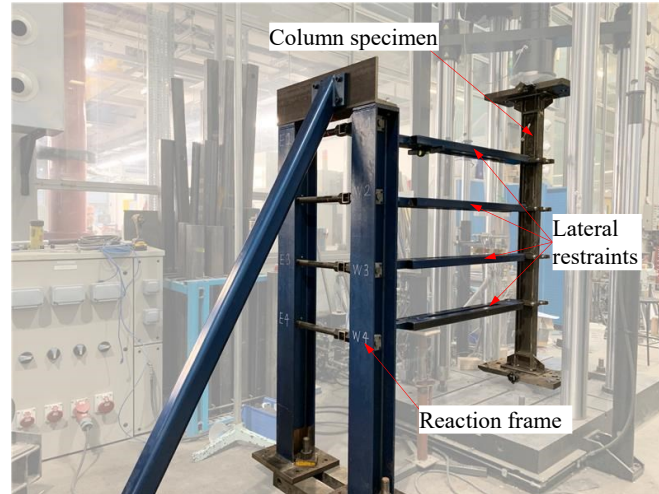


Figure 3.9: Restraint system for major axis flexural buckling tests

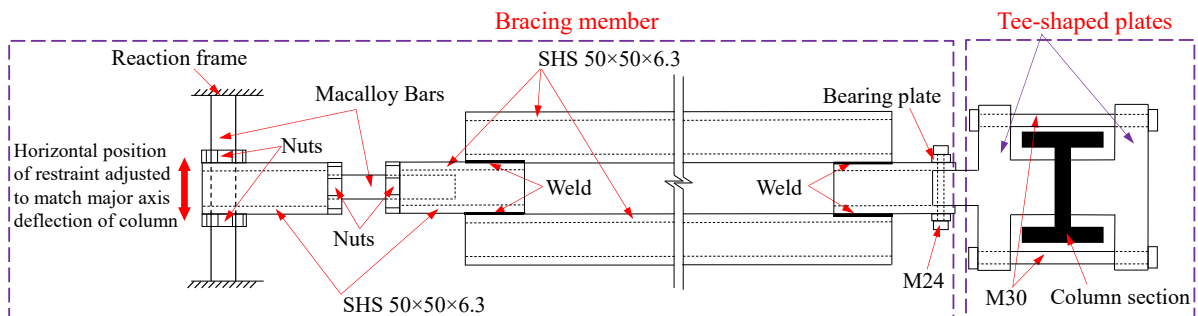
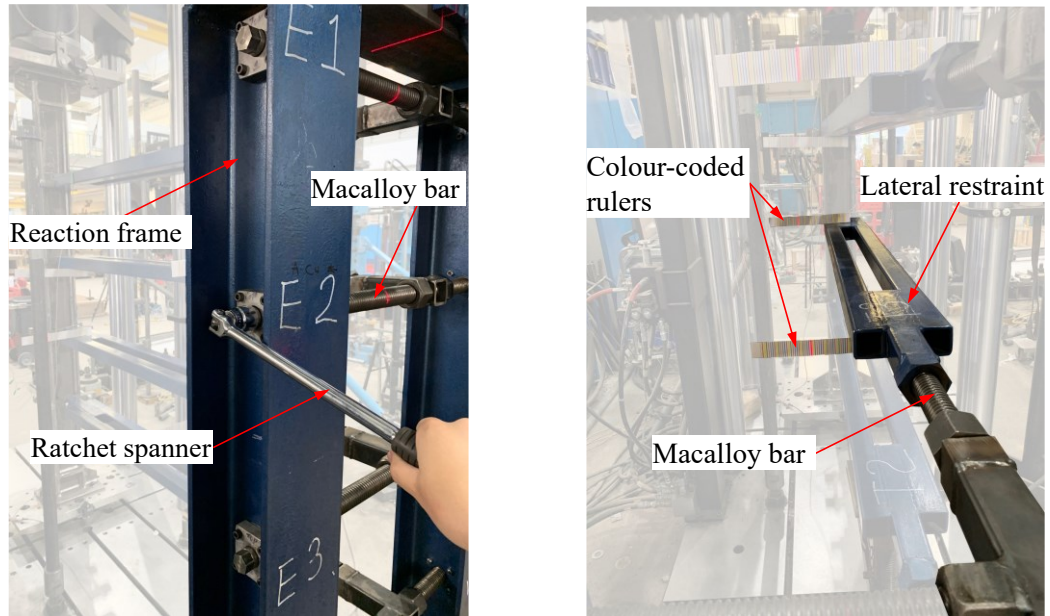


Figure 3.10: Schematic drawing of lateral restraint system used in flexural buckling tests

During testing, the horizontal positions of the lateral restraints were continuously adjusted by means of a ratchet spanner to match the lateral deflections of the column specimens at the corresponding heights, as shown in Figure 3.11(a). To ensure alignment of the lateral restraints and the column specimens throughout the tests, a pair of rulers with multiple coloured lines at intervals of 2 mm were affixed to both ends of each lateral restraint to serve as a reference for the adjustments; the horizontal positions of the lateral restraints were adjusted according to a laser line projected vertically onto the colour coded rulers, as shown in Figure 3.11(b).

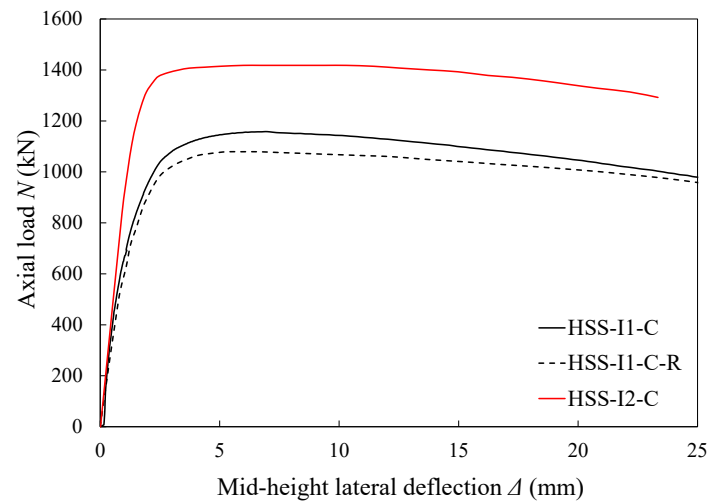


(a) Adjusting horizontal position of lateral restraints using a ratchet spanner

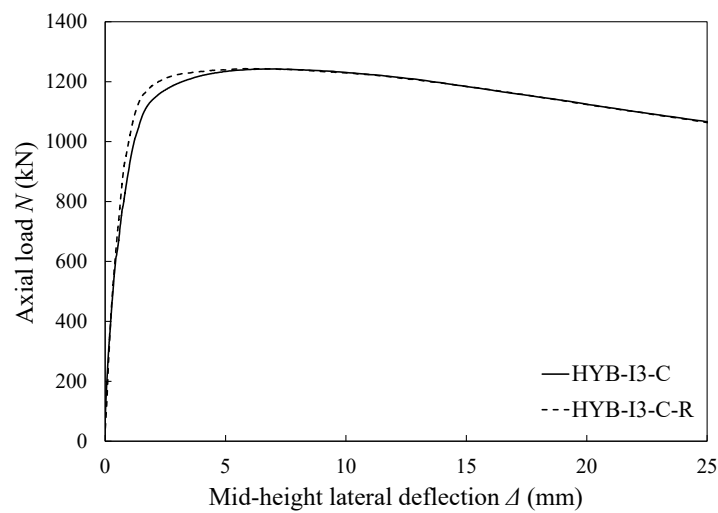
(b) Laser line projected onto colour-coded rulers

Figure 3.11: Adjustment of horizontal position of lateral restraints during testing

The experimental axial load–mid-height lateral deflection ($N-\Delta$) curves for the homogeneous and hybrid welded I-section columns are presented in Figure 3.12(a) and (b), respectively. The ultimate major axis flexural buckling load N_u and the corresponding mid-height lateral deflection δ_u for each column specimen are summarised in Table 3.5. The failure modes of all tested columns are shown in Figure 3.13; all column specimens buckled in the direction dictated by the global imperfections. As shown in Figure 3.13, the failure of all column specimens was dominated by global buckling with no pronounced local buckling due to the compact nature of the chosen cross-section profiles.



(a) Homogeneous welded I-section columns



(b) Hybrid welded I-section columns

Figure 3.12: Experimental axial load–mid-height lateral deflection curves



Figure 3.13: Failure modes of tested welded I-section columns

3.4 Numerical simulations

3.4.1 General

In conjunction with the experimental efforts, numerical investigations were performed with the aims of developing reliable finite element (FE) models to replicate the test results and then carrying out comprehensive parametric studies to generate supplementary flexural buckling resistance data for both homogeneous and hybrid welded I-section columns covering a wide range of cross-section dimensions, member slenderness and material grades. Details of the numerical investigations are provided in the present section.

3.4.2 Development of FE models

The numerical simulations for the welded I-section columns were conducted using the general-purpose FE package Abaqus version 6.21 (2021). All the developed FE models in the present study were meshed with the S4R element (i.e. a four-noded shell element with reduced integration from the Abaqus element library (ABAQUS, 2021)), which has been successfully applied in previous numerical investigations of I-section structural components subjected to different loading conditions (Bu & Gardner, 2019b; Feng et al., 2020; Russell et al., 2020; Su et al., 2021; Tse et al., 2021; Yun et al., 2018b). For validation purposes, the measured cross-sectional geometries of welded I-sections including the weld fillets were carefully modelled. The weld fillets were assumed to be symmetric and represented by five web elements with equal height but different widths (i.e. thicknesses), as illustrated in Figure 3.14. In order to avoid any overlap between the flange and web plates, nodes at both edges of the web were offset from the web-to-flange junctions by half the flange thickness, and coupled to their corresponding nodes at the mid-thickness of the flange plates using *MPC BEAM constraints, as depicted in Figure 3.14.

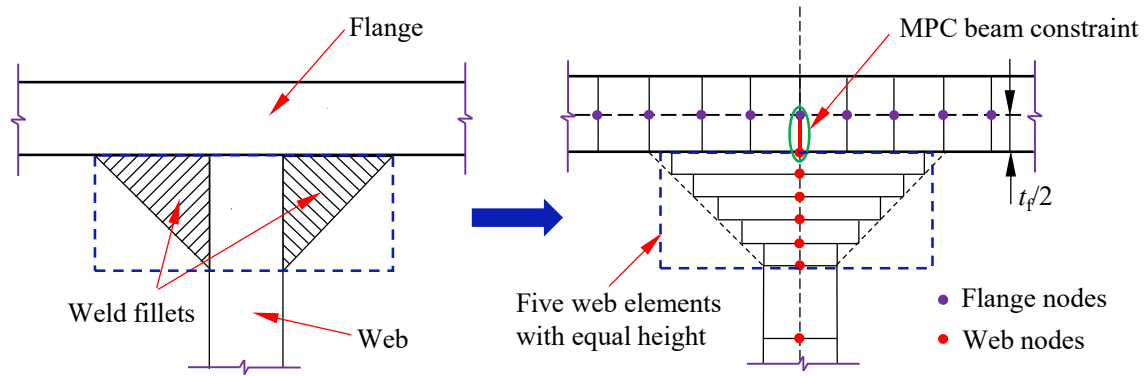


Figure 3.14: Modelling technique for geometrical representation of weld fillets in welded I-sections

Following a mesh sensitivity investigation, an element size approximately equal to $(B+H)/40$ was selected to discretise the modelled column specimens in both the transverse and longitudinal directions; the adopted mesh density was shown to provide sufficiently accurate results with reasonable computational efficiency. The measured engineering stress-strain curves were transformed into true stress-logarithmic plastic strain curves before input into Abaqus.

Both initial local and global geometric imperfections were introduced into the FE models by modifying the nodal coordinates of the original perfect geometry. Local geometric imperfections were included in the form of a series of sinusoidal waves (see Figure 3.15) with their half-wavelength approximately equal to the elastic local buckling half-wavelength of the modelled welded I-section in compression $L_{b,cs}$, which can be determined numerically (e.g. using the readily available finite strip software CUFSM (Schafer & Adany, 2006)) or approximated using analytical expressions developed by Fieber et al. (2019b); the former was employed herein. The local imperfection amplitude of the web δ_w was taken as $1/200$ of the web height h_w (i.e. $h_w = H - 2t_f - 2t_{weld}$) when the web plate was more susceptible to local buckling than the flange plates (i.e. when the web plate had a higher plate slenderness $\bar{\lambda}_{p,w}$ than that of the flange plates $\bar{\lambda}_{p,f}$ determined in accordance with EN 1993-1-5 (2006)), otherwise the local imperfection amplitude of the flange δ_f was taken as $1/50$ of the clear width of the flange outstand b_f (i.e. $b_f = (B - t_w)/2 - t_{weld}$). The flange plate slenderness $\bar{\lambda}_{p,f}$ and web plate slenderness $\bar{\lambda}_{p,w}$ were determined from Equations (3.6) and (3.7), respectively:

$$\bar{\lambda}_{p,f} = \sqrt{\frac{f_{y,f}}{\sigma_{cr,f}}} = \frac{b_f / t_f}{28.4 \varepsilon_f \sqrt{k_\sigma}} \quad (3.6)$$

$$\bar{\lambda}_{p,w} = \sqrt{\frac{f_{y,w}}{\sigma_{cr,w}}} = \frac{h_w / t_w}{28.4 \varepsilon_w \sqrt{k_\sigma}} \quad (3.7)$$

in which $\sigma_{cr,f}$ and $\sigma_{cr,w}$ are the elastic local buckling stresses of the isolated flange and web plates, respectively, assuming simply-supported boundary conditions at the edges of the adjoining plates, $\varepsilon_f = (235/f_{y,f})^{0.5}$ and $\varepsilon_w = (235/f_{y,w})^{0.5}$ are the flange and web material parameters, respectively, and k_σ is the elastic buckling factor that is equal to 0.43 for the isolated outstand flange under uniform compression and 4 for the isolated internal web under uniform compression, respectively, in accordance with EN 1993-1-5 (2006). Once the local imperfection amplitude of the critical plate was known, the corresponding amplitude for the non-critical plate was determined based on the assumption that the angle of the web-to-flange junctions remained at 90°. The adopted local imperfection amplitudes correspond to the fabrication tolerances for welded I-section members according to EN 1090-2 (2018).

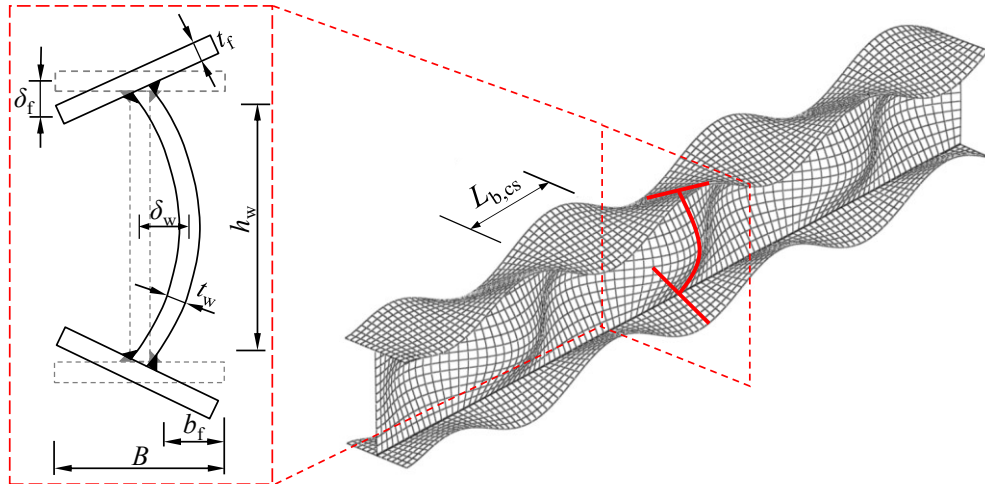


Figure 3.15: Form of local geometric imperfections (not to scale) employed in FE models

Global geometric imperfections about the buckling axis were assigned to the FE models in the shape of a half-sine wave along the longitudinal direction of the column. A total of four global

imperfection amplitudes, namely the measured global imperfection e_g and three generalised values of $L_{cr}/1000$, $L_{cr}/1500$ and $L_{cr}/2000$, were considered in order to investigate the sensitivity of the FE models to variations in the global imperfection amplitudes. The Abaqus input files containing the nodal coordinates with the pre-defined local and global imperfections were generated using the software package Matlab (2021).

The proposed residual stress pattern for welded I-sections, as described in Section 3.2 of the present chapter, was employed in the FE models; the residual stresses were taken to be constant through the thickness (i.e. membrane stresses) and simulated explicitly as an initial stress condition by means of the Abaqus `*INITIAL CONDITIONS` command. To illustrate the impact of the residual stress amplitudes on the flexural buckling resistances of welded I-section columns, both the mean and upper characteristic values of the residual stresses (denoted as “RS_m” and “RS_uc”, respectively), as determined using Equations (3.1) and (3.2) respectively, were considered in the FE models.

With regards to the boundary conditions, all nodes at each end section of the column specimens were coupled to a reference point, positioned on the centroid axis of the cross-section at the location of the corresponding knife edge, by means of kinematic coupling constraints. Pinned in-plane and fixed out-of-plane boundary conditions were then applied to the reference points. Note that for the simulation of the major axis flexural buckling tests carried out in the present study, the out-of-plane displacement degree of freedom at the web-to-flange junction was also restrained at the cross-sections where the lateral restraints were provided, as shown in Figure 3.9, thus eliminating the influence of out-of-plane buckling which is beyond the scope of the present study.

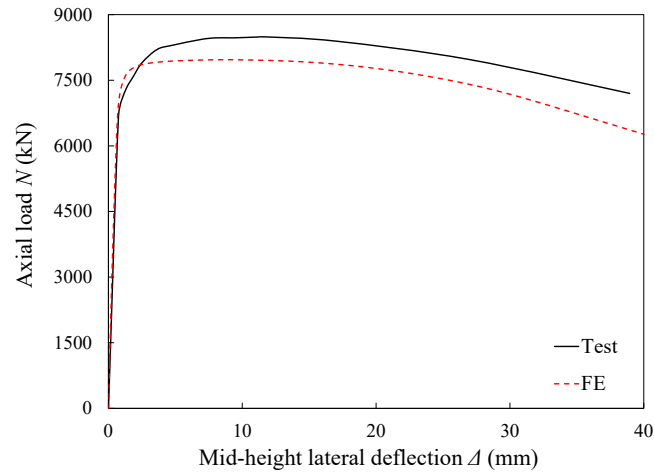
3.4.3 Validation of FE models

The developed FE models were validated against the experimental results from the minor axis flexural buckling tests on homogeneous welded I-section columns reported in (Ban et al., 2013b; Deng et al., 2021; Li et al., 2016) and the major axis flexural buckling tests on both homogeneous and hybrid welded I-section columns described in Section 3.3. Table 3.6 presents the ratios of the ultimate resistances obtained from the physical tests $N_{u, test}$ to the ultimate

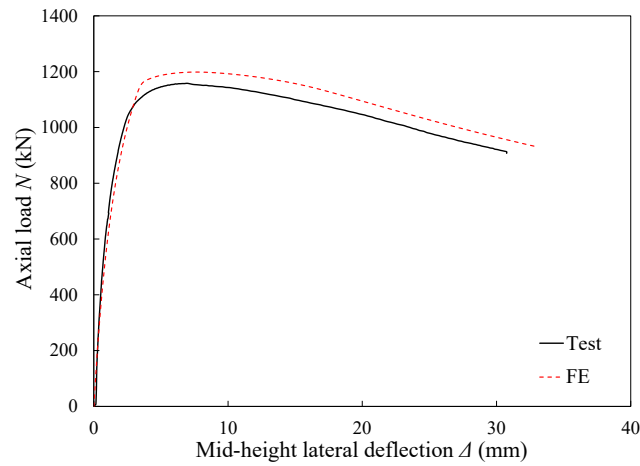
resistances derived from the FE models $N_{u,FE}$ considering the different combinations of the amplitudes of global geometric imperfections and residual stresses. The statistical mean and COV (i.e. coefficient of variation) values of the $N_{u,test}/N_{u,FE}$ ratios for the column specimens buckling about the major axis and about the minor axis, as well as for all specimens, are presented in Table 3.6. The results of the comparisons show that the best agreement between the test and FE ultimate resistances was achieved when the combination of the measured global imperfection amplitude and the mean value of the residual stresses (i.e. RS_m) was employed. Using the combination of $L_{cr}/1000$ and RS_m in the FE models also results in overall accurate and consistent predictions of the test ultimate resistances, though the numerical resistance predictions for column specimens buckling about the minor axis were slightly conservative since the global imperfection amplitudes of $L_{cr}/1000$ are generally greater than the measured values (Ban et al., 2013b; Deng et al., 2021; Li et al., 2016). The experimental axial load–mid-height lateral deflection curves were well replicated by the FE models using the amplitude combination of $L_{cr}/1000$ and RS_m, as shown in Figure 3.16 for three representative welded I-section column specimens.

Table 3.6: Comparisons of welded I-section column test results with FE results for varying combinations of global geometric imperfection and residual stress amplitudes

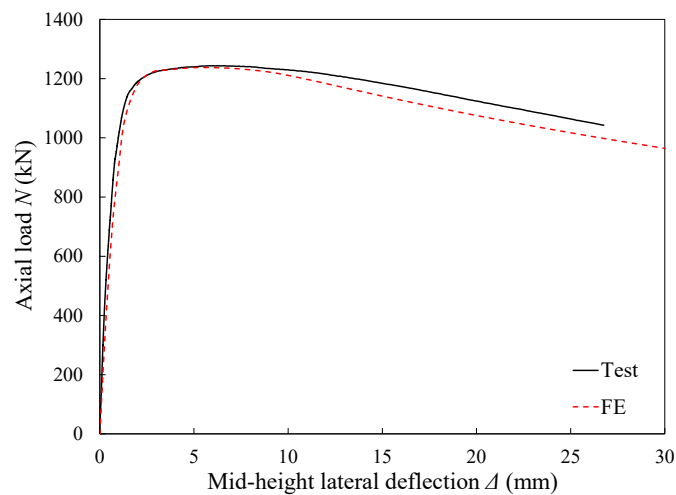
Reference	Cross-section type (steel grade)	Specimen label	Buckling axis	$N_{u,test}/N_{u,FE}$ (Global geometric imperfection and residual stress combinations)							
				RS_m				RS_uc			
				Measured	$L_{cr}/1000$	$L_{cr}/1500$	$L_{cr}/2000$	Measured	$L_{cr}/1000$	$L_{cr}/1500$	$L_{cr}/2000$
Deng et al. (2021)	Homogeneous (Q460GJ)	H36	Minor axis	1.04	1.06	1.05	1.04	1.09	1.11	1.10	1.09
		H43		1.10	1.14	1.12	1.11	1.17	1.21	1.19	1.18
		H50		0.98	1.01	0.98	0.97	1.06	1.08	1.06	1.04
		H57		1.05	1.08	1.04	1.02	1.13	1.16	1.12	1.09
		H64		0.94	1.01	0.96	0.93	1.01	1.07	1.02	1.00
		H72		1.00	1.16	1.09	1.06	1.06	1.22	1.15	1.12
		H78		1.02	1.09	1.03	1.00	1.06	1.14	1.07	1.04
		H86		0.94	1.02	0.97	0.94	0.96	1.06	1.00	0.96
Li et al. (2016)	Homogeneous (Q690)	H-30-1		1.07	1.07	1.05	1.05	1.12	1.12	1.11	1.10
		H-30-2		1.08	1.12	1.10	1.09	1.15	1.18	1.16	1.16
		H-50-1		1.08	1.17	1.13	1.10	1.15	1.24	1.20	1.17
		H-50-2		1.07	1.15	1.11	1.09	1.15	1.22	1.18	1.16
		H-70-2		0.88	0.96	0.92	0.89	0.90	0.99	0.94	0.91
Ban et al. (2013b)	Homogeneous (Q960)	H2-960		1.02	0.95	0.91	0.89	1.04	0.98	0.94	0.91
		H3-960		1.08	1.08	1.06	1.05	1.08	1.08	1.06	1.05
Mean for column specimens buckling about the minor axis				1.02	1.07	1.03	1.02	1.08	1.12	1.09	1.07
COV for column specimens buckling about the minor axis				0.06	0.06	0.07	0.07	0.07	0.07	0.08	0.08
Present study	Hybrid (Q690 + Q355)	HSS-I1-C	Major axis	0.98	0.97	0.94	0.93	1.00	1.03	1.00	0.99
		HSS-I1-C-R		0.94	0.94	0.92	0.90	1.03	1.02	0.98	0.97
		HSS-I2-C		0.95	0.93	0.91	0.90	1.00	0.98	0.96	0.96
		HYB-I3-C		1.00	1.00	0.99	0.98	1.02	1.02	1.03	1.02
		HYB-I3-C-R		1.01	1.00	0.99	0.98	1.03	1.03	1.03	1.02
Mean for column specimens buckling about the major axis				0.98	0.97	0.95	0.94	1.02	1.01	1.00	0.99
COV for column specimens buckling about the major axis				0.03	0.04	0.04	0.04	0.02	0.02	0.03	0.03
Mean for all column specimens				1.01	1.04	1.01	0.99	1.06	1.10	1.06	1.05
COV for all column specimens				0.06	0.07	0.07	0.08	0.07	0.08	0.08	0.08



(a) Q690 steel homogeneous welded I-section column buckling about the minor axis (Specimen H-30-1 tested in (Li et al., 2016))



(b) S690 steel homogeneous welded I-section column buckling about the major axis (Specimen HSS-I1-C)



(c) Hybrid welded I-section column buckling about the major axis (Specimen HYB-I3-C)

Figure 3.16: Comparisons of typical test and FE load-deformation curves for welded I-section columns

It may be observed from Table 3.6 that the amplitude of the residual stresses has a clear influence on the ultimate resistances of the welded I-section columns. The increase in residual stress amplitude (i.e. from RS_m to RS_uc) is more detrimental to the welded I-section columns buckling about the minor axis due to the added flange compressive residual stresses promoting earlier yielding at the flange tips, which make the greatest contribution to the member stability. Use of the RS_uc residual stresses in the FE models resulted in an average decrease in capacity of almost 5% in comparison to that obtained from the FE models with the RS_m residual stresses.

To conclude, the developed FE models using the combination of measured (or $L_{cr}/1000$ if measurements are unavailable) geometric imperfection amplitudes and RS_m residual stresses have been shown to be able to simulate the observed behaviour of welded I-section columns accurately and are suitable for FE model validation. For parametric studies, on the other hand, where more safe-sided results are typically sought, use of the combination of $L_{cr}/1000$ geometric imperfection amplitudes and RS_uc residual stresses is recommended.

3.4.4 Numerical parametric studies

Following the successful validation of the developed FE models, comprehensive numerical parametric studies were performed with the aim of expanding the data pool for both homogeneous and hybrid welded I-section columns over a wider variety of steel grades, cross-sectional geometries and member slendernesses. Five hot-rolled steel grades were considered in the parametric studies: S235, S355, S460, S690 and S960, leading to a total of five different homogeneous welded I-sections and five different hybrid welded I-sections (i.e. I-sections made of (1) S460 steel flanges and an S235 steel web, (2) S460 steel flanges and an S355 steel web, (3) S690 steel flanges and an S355 steel web, (4) S690 steel flanges and an S460 steel web and (5) S960 steel flanges and an S690 steel web) being investigated. The bilinear plus nonlinear hardening material model developed by Yun and Gardner (2017) was used to derive the full-range stress-strain curves for the S235, S355, S460 and S690 hot-rolled steels. Owing to the fact that only a limited number of experimental stress-strain curves on ultra-high strength S960 steel were available to underpin the Yun and Gardner model, the average measured stress-

strain curve obtained from the longitudinal tensile coupon tests on S960 steel reported in (ABAQUS, 2021) was adopted to simulate the S960 steel plates considered herein. The employed full-range stress-strain curves for the five studied steel grades are shown in Figure 3.17, and the corresponding basic mechanical properties (E , f_y and f_u) are provided in Table 3.7. Note that the basic mechanical properties for the S235, S355, S460 and S690 hot-rolled steels are taken in accordance with the latest version of Eurocode 3 (EC3) – prEN 1993-1-1 (2019), while the additional material parameters (e.g. ε_{sh} and ε_u) which are used to form the stress-strain curves, were determined from the predictive expressions given in Yun and Gardner (2017).

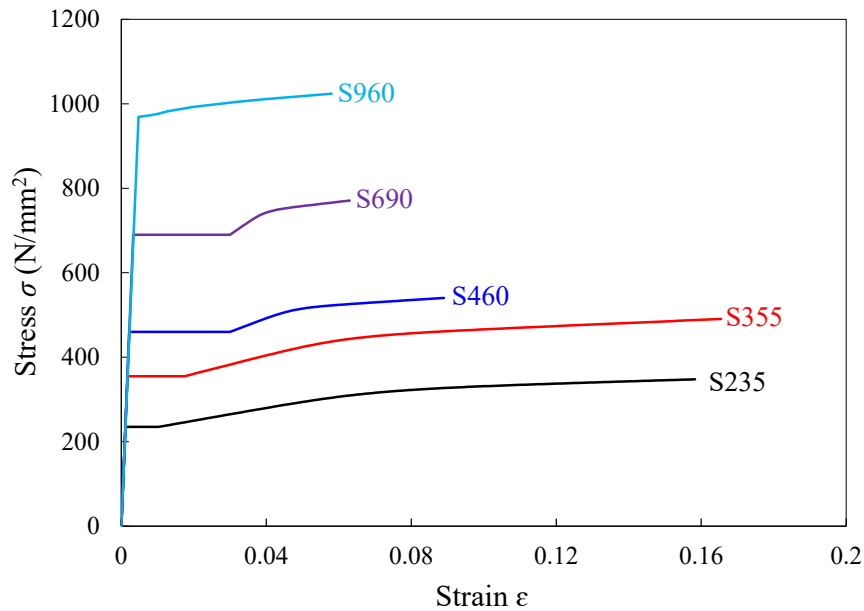


Figure 3.17: Full-range stress-strain curves for five hot-rolled steel grades employed in parametric studies

Table 3.7: Basic mechanical properties for five hot-rolled steel grades employed in parametric studies

Steel grade	E N/mm ²	f_y N/mm ²	f_u N/mm ²
S235	210000	235	360
S355	210000	355	490
S460	210000	460	540
S690	210000	690	770
S960	204393	969	1024

Regarding the welded I-section geometries, a constant flange width B of 100 mm was adopted in the numerical parametric studies, while the cross-section height H was varied to obtain three cross-section aspect ratios H/B of 1.0, 1.5 and 2.0. For each cross-section aspect ratio, five different values of flange thickness t_f (ranging from 5 mm to 20 mm) and web thickness t_w (ranging from 2.8 mm to 28 mm) were employed to generate a broad spectrum of cross-section slendernesses within the Class 1-3 domain according to prEN 1993-1-1 (2019). The t_f and t_w values for all simulated sections were specified such that $\bar{\lambda}_{p,f} \approx \bar{\lambda}_{p,w}$, thus minimising the influence of element interaction between the flanges and web on the local buckling behaviour of the sections (Gardner et al., 2019). The fillet welds at the web-flange intersections were ignored for all the modelled welded I-section columns in both the parametric studies and, for consistency, the subsequent analysis and assessment/establishment of design provisions. Columns with twenty different lengths were simulated for each cross-section to achieve a broad range of non-dimensional member slenderness $\bar{\lambda}$ from 0.2 to 2.5. The tolerance-based local geometric imperfection amplitudes, together with the amplitude combination of $L_{cr}/1000$ (i.e. the amplitude of global geometric imperfections) and RS_{uc} (i.e. the amplitude of the residual stresses), were employed throughout the parametric studies. For the column specimens buckling about the major axis, out-of-plane lateral displacements were restrained at the web-flange junction at regular intervals.

A total of 6000 numerical data points on homogeneous and hybrid welded I-sections columns were generated, covering flexural buckling about both the major and minor axes, as well as various steel grades, cross-section geometries and member slendernesses. The numerically derived data, together with the available experimental results, are used in the next section for the assessment of existing design provisions and development of new design proposals for both homogeneous and hybrid welded I-section columns.

3.5 Evaluation of the current design codes and new design proposals

3.5.1 General

In this section, the applicability and accuracy of the relevant column buckling design provisions, as set out in prEN 1993-1-1 (2019) and AISC 360 (2016), are assessed for the design of both homogeneous and hybrid welded I-section columns made of varying steel grades. The assessment was made by comparing the ultimate loads obtained from the tests and FE simulations $N_{u,\text{test/FE}}$ to the unfactored resistance predictions determined according to prEN 1993-1-1 ($N_{u,\text{EC3}}$) or AISC 360 ($N_{u,\text{AISC}}$). Shortcomings in the current design rules are revealed; hence, new design proposals to overcome the identified shortcomings are developed.

3.5.2 European code prEN 1993-1-1 (EC3)

The latest version of prEN 1993-1-1 (EC3) (2019) specifies different column buckling curves based on the Ayrton-Perry type formulation (Ayrton & Perry, 1886) for the design of structural steel column members with material grades up to S700 susceptible to global instability (i.e. flexural buckling, torsional buckling and flexural-torsional buckling). The flexural buckling resistance $N_{b,\text{EC3,Rd}}$ of welded I-section columns with non-slender cross-sections (i.e. Class 1-3 cross-sections) is given by Equation (3.8):

$$N_{b,\text{EC3,Rd}} = \frac{\chi_{\text{EC3}} A f_y}{\gamma_{\text{M1}}}, \text{ for non-slender welded I-section columns} \quad (3.8)$$

in which γ_{M1} is the partial safety factor for member buckling, with a recommended value of 1.0 (EN 1993-1-1, 2019), and χ_{EC3} is the EC3 flexural buckling reduction factor, as given by Equations (3.9) and (3.10):

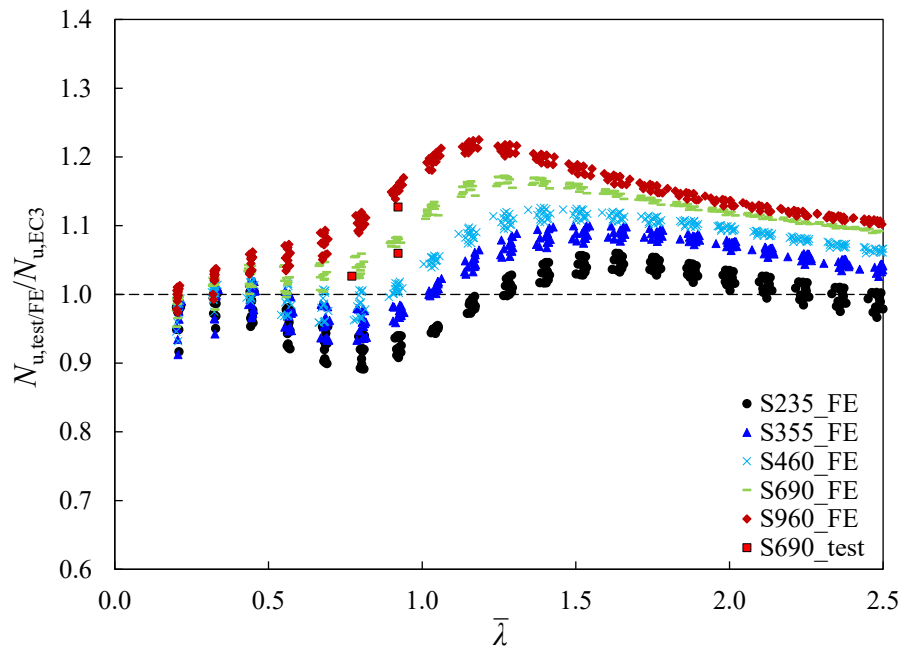
$$\chi_{\text{EC3}} = \frac{1}{\phi_{\text{EC3}} + \sqrt{\phi_{\text{EC3}}^2 - \bar{\lambda}^2}} \text{ but } \chi_{\text{EC3}} \leq 1.0 \quad (3.9)$$

$$\phi_{\text{EC3}} = 0.5 \left(1 + \alpha_{\text{EC3}} \left(\bar{\lambda} - \bar{\lambda}_{0,\text{EC3}} \right) + \bar{\lambda}^2 \right) \quad (3.10)$$

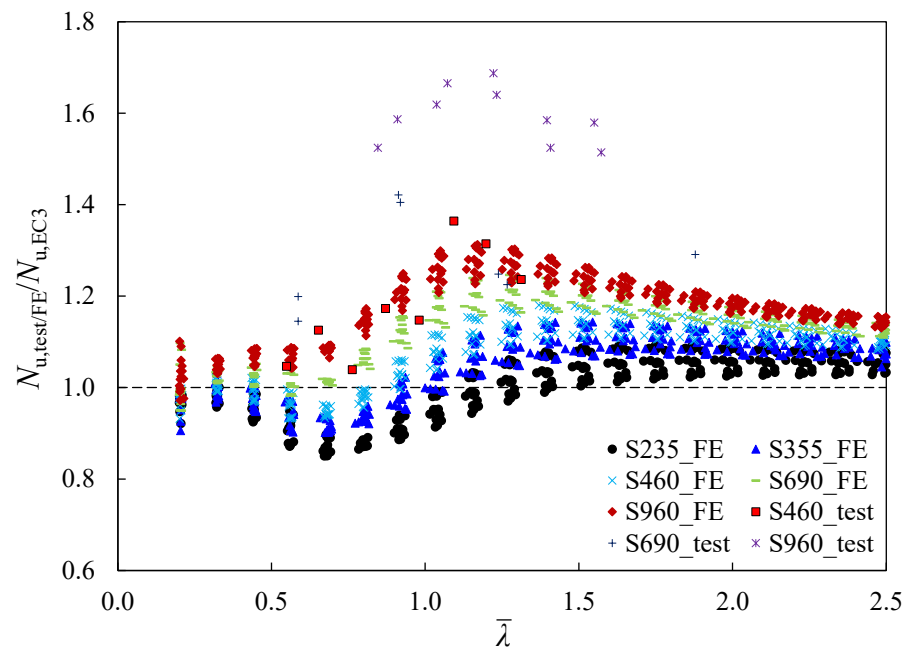
where $\bar{\lambda}$ is the non-dimensional member slenderness given by Equation (3.3) and α_{EC3} (i.e. the imperfection factor) and $\bar{\lambda}_{0,EC3}$ (i.e. the plateau length of the buckling curve) are parameters that account for the combined effects of geometric imperfections and residual stresses on the flexural buckling resistances of column members; the values of α_{EC3} and $\bar{\lambda}_{0,EC3}$ for different cross-section shapes and design parameters are specified in prEN 1993-1-1 (2019). Specifically, prEN 1993-1-1 (2019) adopts buckling curve “c” (with $\bar{\lambda}_{0,EC3} = 0.2$ and $\alpha_{EC3} = 0.49$) for homogeneous welded I-section columns with $t_f \leq 40$ mm buckling about the minor axis and buckling curve “b” (with $\bar{\lambda}_{0,EC3} = 0.2$ and $\alpha_{EC3} = 0.34$) for homogeneous welded I-section columns with $t_f \leq 40$ mm buckling about the major axis for all grades of steel.

The accuracy and suitability of the EC3 column buckling curves for both homogeneous and hybrid welded I-section columns made of varying steel grades are assessed based on the established test and FE data. Note that the yield strength f_y is replaced by the weighted average yield strength $f_{y,a}$ given by Equation (3.4) for the determination of the non-dimensional member slenderness $\bar{\lambda}$ (using Equation (3.3)) and the EC3 flexural buckling resistance (using Equation (3.8)) of the hybrid welded I-section columns. The test and FE ultimate resistances $N_{u,test/FE}$ are normalised by the unfactored EC3 resistance predictions $N_{u,EC3}$ and plotted against the column non-dimensional member slenderness $\bar{\lambda}$ (i.e. $\bar{\lambda} = (Af_y/N_{cr})^{0.5}$ for the homogeneous welded I-section columns and $\bar{\lambda} = (Af_{y,a}/N_{cr})^{0.5}$ for the hybrid welded I-section columns) in Figure 3.18. It can be seen from Figure 3.18 that the EC3 design approach yields flexural buckling resistance predictions with varying levels of accuracy for columns made of different steel grades. The EC3 resistance predictions are generally conservative for welded I-sections columns made of HSS grades (i.e. S690 steel and S960 steel), while a number of EC3 resistance predictions appear on the unsafe side for columns made of NSS grades (i.e. homogeneous welded I-sections made of S235, S355 and S460 steels, and hybrid welded I-sections made of S460 steel flanges and an S235 steel web as well as S460 steel flanges and an S355 steel web) with intermediate to low non-dimensional member slenderness values. The resistance predictions are also rather scattered, which is also evident from the statistical results of the ratios of $N_{u,test/FE}/N_{u,EC3}$ presented in Table 3.8. The high level of scatter of the EC3 design provisions can be principally attributed to the fact that only a single column buckling curve is employed for the design of welded I-section columns buckling about each axis with different steel grades (i.e. curve “c” for flexural buckling about the minor axis and curve “b” for flexural buckling

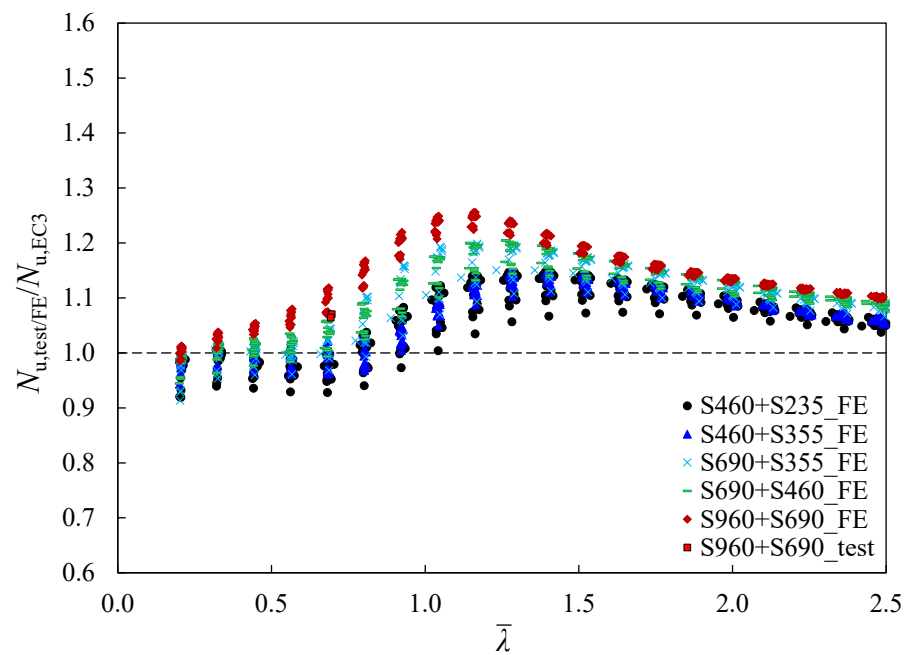
about the major axis), which fails to capture the trend of the reducing relative influence of imperfections, both residual stresses and global geometric imperfections, on the flexural buckling resistances of columns with increasing steel grades (Meng & Gardner, 2020a; Yun et al., 2022). This shortcoming was addressed in a recent research project named Stronger Steels in the Built Environment (STROBE) (Baddoo et al., 2021) where higher flexural buckling curves were proposed for welded I-sections made of S460 steel and above (i.e. using curve “b” for flexural buckling about the minor axis and curve “a” for flexural buckling about the major axis). The ratios of the test and FE column flexural buckling resistances to the resistance predictions using the proposed design curves in STROBE $N_{u,test/FE}/N_{u,STROBE}$ are presented in Table 3.8. It can be seen from Table 3.8 that the proposed design curves in STROBE yield improved resistance predictions in terms of both accuracy and consistency compared to the existing EC3 design provisions, though with scope for further improvement.



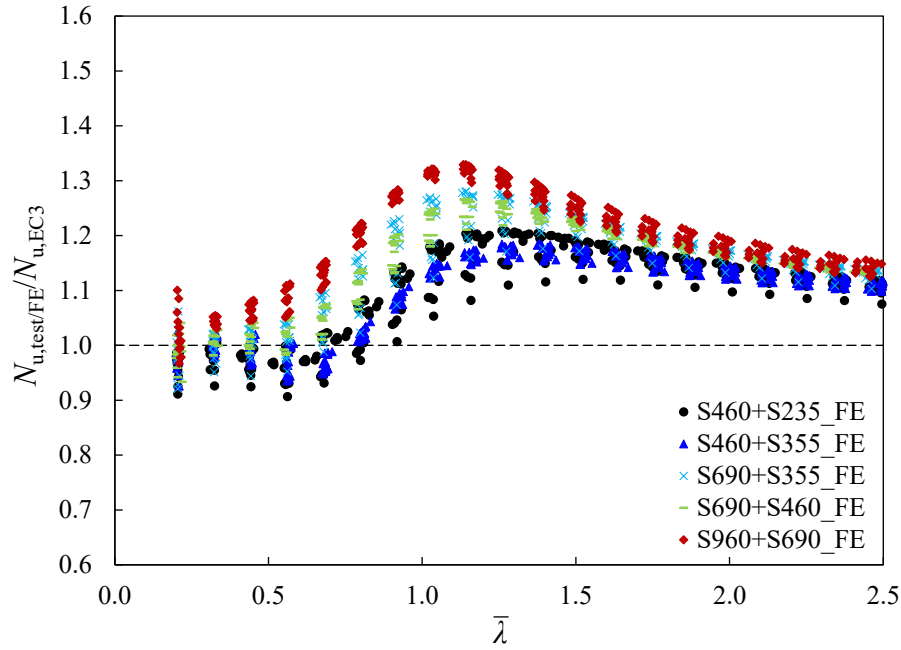
(a) Homogeneous welded I-section columns buckling about the major axis



(b) Homogeneous welded I-section columns buckling about the minor axis



(c) Hybrid welded I-section columns buckling about the major axis



(d) Hybrid welded I-section columns buckling about the major axis

Figure 3.18: Comparisons of test and FE ultimate resistances of column specimens with those predicted using prEN 1993-1-1 (2019)**Table 3.8:** Comparisons of test and FE ultimate resistances of column specimens with those predicted using different design methods

Column type	Buckling axis	Number of data points FE (test)	Evaluation parameter	prEN 1993-1-1 (2019)	STROBE proposal (Baddoo et al., 2021)	AISC 360 (2016)	Modified EC3
				$N_{u,test/FE}/N_{u,EC3}$	$N_{u,test/FE}/N_{u,STROBE}$	$N_{u,test/FE}/N_{u,AISC}$	$N_{u,test/FE}/N_{u,mod-EC3}$
Homogeneous welded I-section	Major	1500 (3)	Mean	1.061	1.017	0.998	1.071
			COV	0.070	0.049	0.060	0.039
	Minor	1500 (35)	Mean	1.088	1.042	0.956	1.062
			COV	0.104	0.079	0.087	0.062
Hybrid welded I-section	Major	1500 (2)	Mean	1.091	1.018	1.026	1.075
			COV	0.069	0.050	0.045	0.047
	Minor	1500 (0)	Mean	1.132	1.056	0.993	1.089
			COV	0.080	0.065	0.054	0.054

3.5.3 American specification AISC 360 (AISC)

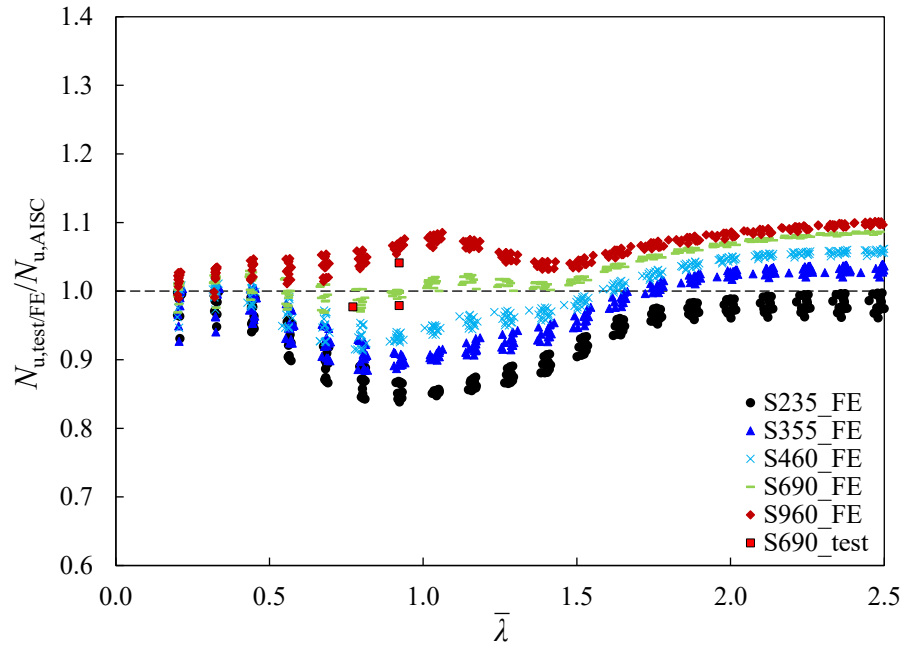
The American Specification AISC 360 (2016) employs the following equation for the determination of the flexural buckling resistance $N_{b,AISC,Rd}$ of homogeneous welded I-section columns with nominal yield strengths up to 690 N/mm²:

$$N_{b,AISC,Rd} = \phi_c \chi_{AISC} A f_y, \text{ for non-slender welded I-section columns} \quad (3.11)$$

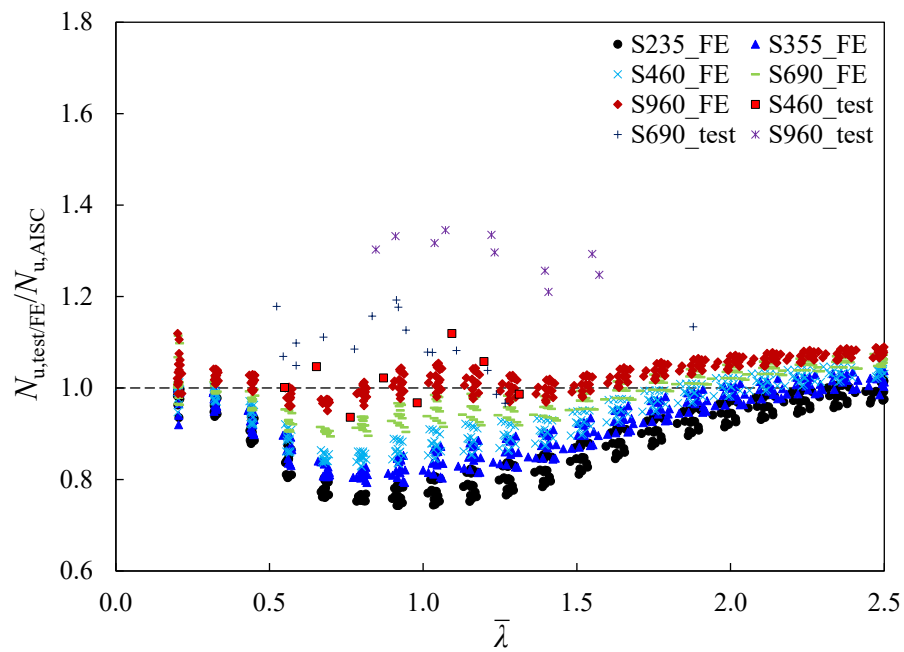
in which ϕ_c is the resistance factor for member buckling resistances (equivalent to the inverse of the EC3 partial safety factor γ_{M1} in Equation (3.8)), which has a recommended value of 0.9 in (AISC 360, 2016), and χ_{AISC} is the AISC flexural buckling reduction factor, which can be determined from a two-stage column buckling curve as expressed by Equation (3.12).

$$\chi_{AISC} = \begin{cases} 0.658^{\bar{\lambda}^2} & \text{for } \bar{\lambda} \leq 1.5 \\ \frac{0.877}{\bar{\lambda}^2} & \text{for } \bar{\lambda} > 1.5 \end{cases} \quad (3.12)$$

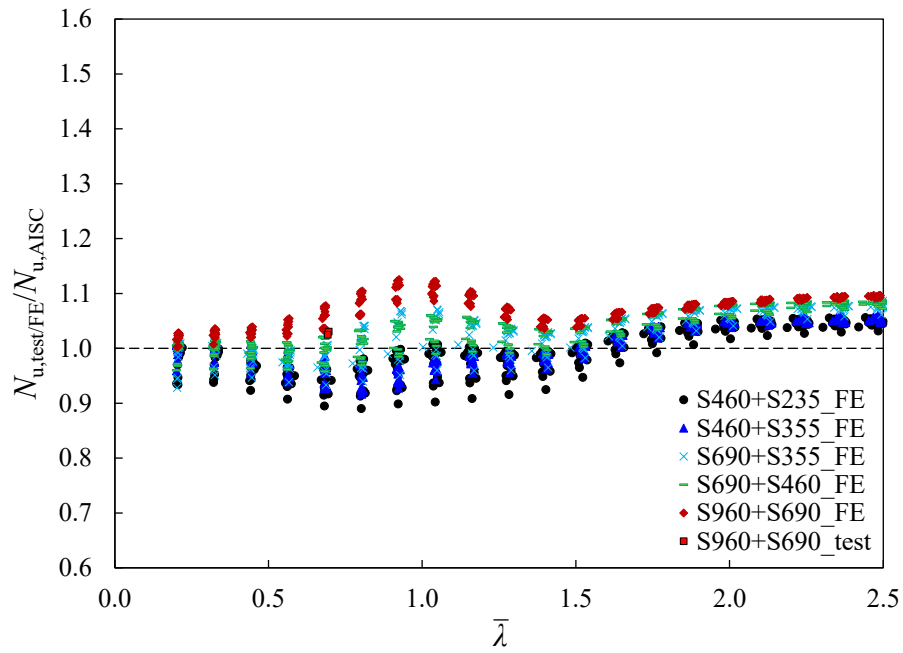
Similar to the EC3 design provisions, the American Specification AISC 360 (2016) does not provide specific design rules for hybrid welded I-section columns failing by in-plane flexural buckling. In this subsection, the applicability of the two-stage column buckling curve to hybrid welded I-section columns is also assessed, with the weighted average yield strength $f_{y,a}$ given by Equation (3.4) used for the calculation of the non-dimensional member slenderness $\bar{\lambda}$ (using Equation (3.3)) and the AISC flexural buckling resistance (using Equation (3.11)). The test and FE ultimate buckling resistances $N_{u,test/FE}$ are compared with the unfactored AISC resistance predictions $N_{u,AISC}$ in Figure 3.19 and Table 3.8. It is shown that although the AISC design method generally leads to accurate resistance predictions for both homogeneous and hybrid welded I-section columns made of HSS grades, it provides unsafe predictions for welded I-section columns made of lower steel grades, especially for those buckling about the minor axis. The varying levels of accuracy of the AISC design method for welded I-section columns with different steel grades would be expected since the AISC 360 (2016) adopts a single column buckling curve for welded I-section columns failing by in-plane flexural buckling about either axis, thus ignoring the influence of both the axis of buckling and the steel grade on the flexural buckling resistance of welded I-section columns.



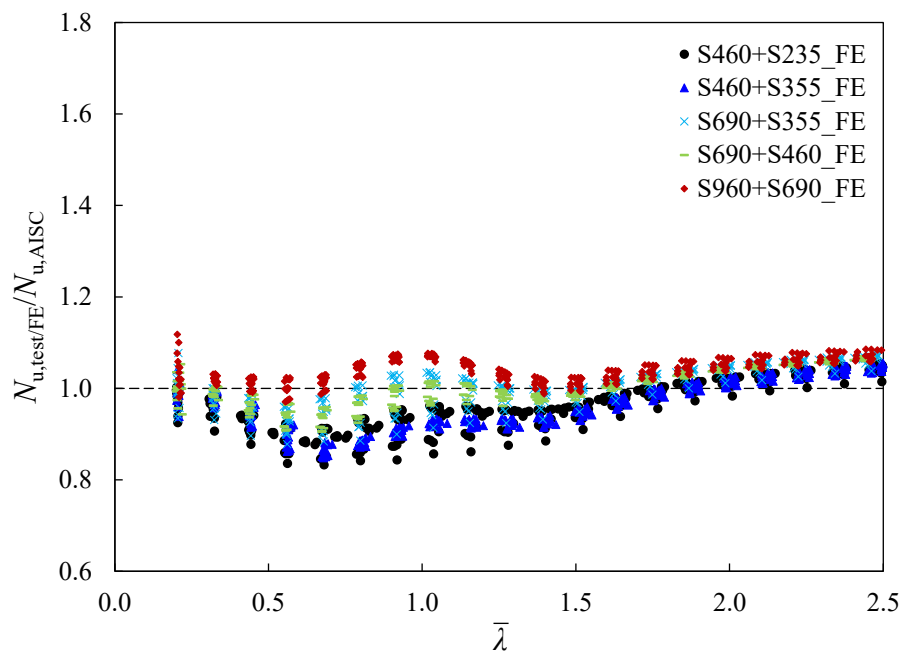
(a) Homogeneous welded I-section columns buckling about the major axis



(b) Homogeneous welded I-section columns buckling about the minor axis



(c) Hybrid welded I-section columns buckling about the major axis



(d) Hybrid welded I-section columns buckling about the minor axis

Figure 3.19: Comparisons of test and FE ultimate resistances of column specimens with those predicted using AISC 360 (2016)

3.5.4 New design proposals

On the basis of the above assessments, it is generally found that the existing design rules of prEN 1993-1-1 (2019) and AISC 360 (2016) have scope for improvement in the flexural buckling resistance prediction of homogeneous and hybrid welded I-section columns made of varying steel grades. Welded I-section columns generally exhibit superior normalised column buckling resistances with increasing yield strength; this is attributed to the reducing relative influence of the initial imperfections, including both the residual stresses and the initial global geometric imperfections. Specifically, the residual stresses in welded I-sections have been found to reduce as a proportion of the yield strength with increasing steel grades, as described in Section 3.2, while the amplitudes of global geometric imperfections for columns with a given non-dimensional member slenderness $\bar{\lambda}$ decrease with increasing yield strength since the initial geometric imperfection is conventionally considered to be proportional to the column member length (e.g. $L/1000$) (Meng & Gardner, 2020a). Therefore, to improve the accuracy and consistency of the design method, the influence of the steel grade (i.e. yield strength) on the flexural buckling resistance of welded I-section columns should be rationally considered.

New proposals for the flexural buckling design of homogeneous and hybrid welded I-section columns are developed herein on the basis of the current EC3 design rules. The design proposals feature modified imperfection factors $\alpha_{\text{mod-EC3}}$ for homogeneous and hybrid welded I-section columns as given by Equations (3.13) and (3.14) for major and minor axis flexural buckling, respectively, and a shortened buckling curve plateau length of $\bar{\lambda}_{0,\text{mod-EC3}} = 0.1$ for both homogeneous and hybrid welded I-section columns buckling about either axes, leading to a modified EC3 flexural buckling reduction factor $\chi_{\text{mod-EC3}}$, as described by Equations (3.15) and (3.16). Note that the modified imperfection factors $\alpha_{\text{mod-EC3}}$ are expressed in term of the flange material parameter $\varepsilon_f = (235/f_{y,f})^{0.5}$ since the flanges are decisive in their contribution to the flexural buckling resistance of welded I-section columns.

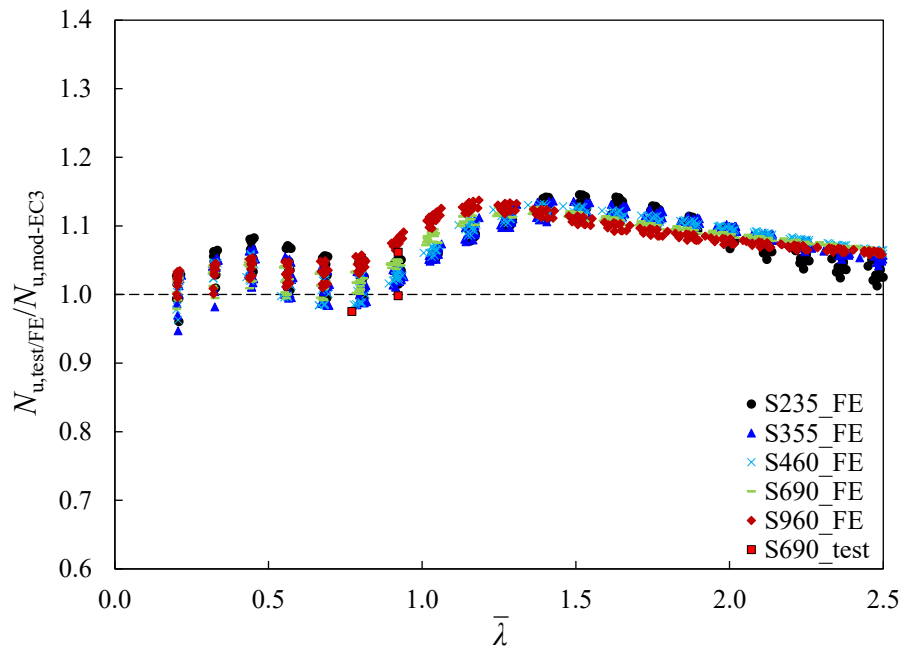
$$\alpha_{\text{mod-EC3}} = 0.45\varepsilon_f, \text{ for flexural buckling about the major axis} \quad (3.13)$$

$$\alpha_{\text{mod-EC3}} = 0.55\varepsilon_f, \text{ for flexural buckling about the minor axis} \quad (3.14)$$

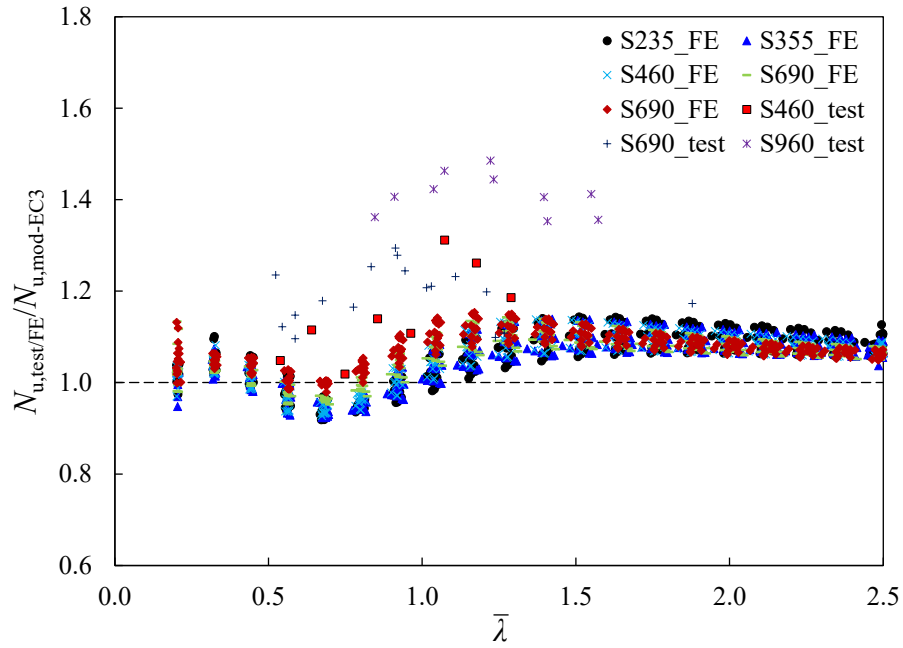
$$\chi_{\text{mod-EC3}} = \frac{1}{\phi_{\text{mod-EC3}} + \sqrt{\phi_{\text{mod-EC3}}^2 - \bar{\lambda}^2}} \text{ but } \chi_{\text{mod-EC3}} \leq 1.0 \quad (3.15)$$

$$\phi_{\text{mod-EC3}} = 0.5 \left(1 + \alpha_{\text{mod-EC3}} \left(\bar{\lambda} - \bar{\lambda}_{0,\text{mod-EC3}} \right) + \bar{\lambda}^2 \right) \quad (3.16)$$

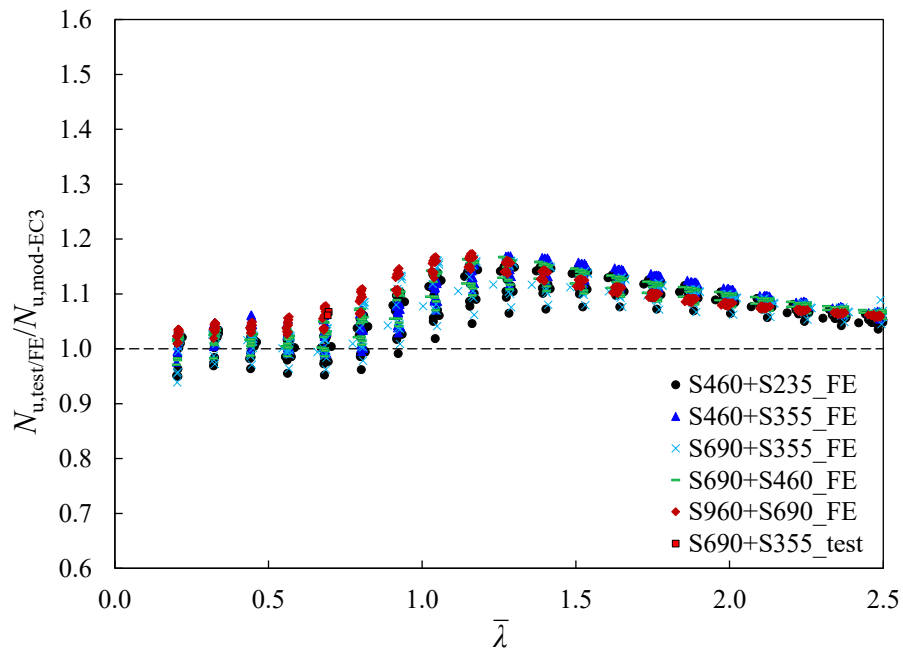
The accuracy of the newly proposed flexural buckling curves for homogeneous and hybrid welded I-section columns is graphically examined in Figure 3.20, in which the ratios of the test and FE column flexural buckling resistances to the resistance predictions using the proposed modified EC3 design approach $N_{u,\text{test/FE}}/N_{u,\text{mod-EC3}}$ are plotted against the non-dimensional member slenderness $\bar{\lambda}$. The normalised data points are seen to display a significantly tighter (i.e. less scattered) trend compared to the EC3 and AISC 360 predictions for both the homogeneous and hybrid welded I-section columns buckling about either axes, indicating that the influence of yield strength on the flexural buckling resistances of welded I-section columns is successfully captured by using the modified flexural buckling curves. The improvements offered by the proposed design rules over the current design methods in terms of accuracy and consistency are also confirmed by the statistical results summarised in Table 3.8.



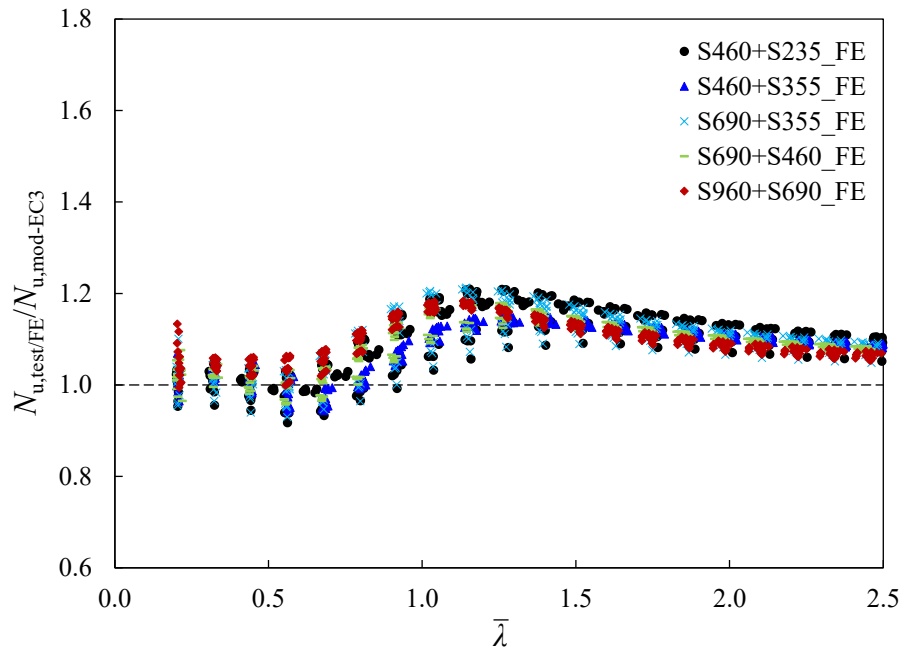
(a) Homogeneous welded I-section columns buckling about the major axis



(b) Homogeneous welded I-section columns buckling about the minor axis



(c) Hybrid welded I-section columns buckling about the major axis



(d) Hybrid welded I-section columns buckling about the minor axis

Figure 3.20: Comparisons of test and FE ultimate resistances of column specimens with those predicted using the new design proposals

3.6 Reliability analysis

In order to quantify the level of reliability of the existing and proposed buckling design approaches for the homogeneous and hybrid welded I-section columns, statistical analyses were performed following the guidance provided in EN 1990 (2002). The variability of the basic geometric and material properties was initially considered. For the welded I-sections, the mean and variability of the key cross-sectional dimensions, including H , B , t_f and t_w , were taken in accordance with Annex E of prEN 1993-1-1 (2019), as presented in Table 3.9, where X_m/X_n is the ratio between the mean and nominal values of a property X and V_X is the corresponding coefficient of variation (COV). The COV of the cross-sectional area A , denoted by V_A , was utilised to represent the combined variability of the cross-sectional dimensions. The value of V_A was derived based on the COV values of the individual dimensions following the procedure set out by Afshan et al. (2015). Note that V_A varies slightly between cross-section profiles, but an average value of $V_A = 0.022$ for the range of considered welded I-sections was employed herein. The mean and variability of the key material properties, i.e. f_y and E , were also taken as those specified in prEN 1993-1-1 (2019), as listed in Table 3.9.

Table 3.9: Mean and COV values of basic material and geometric parameters for welded I-sections in accordance with prEN 1993-1-1 (2019)

Parameter	X_m / X_n	COV
H	1.00	0.009
B	1.00	0.009
t_f	0.98	0.025
t_w	1.00	0.025
f_y (S355, S420)	1.20	0.050
f_y (S460)	1.15	0.045
f_y (above S460)	1.10	0.035
E	1.00	0.030

The combined COV of the geometric and material parameters V_{rt} was subsequently derived as follows. The resistance function was first generalised into Equation (3.17), where the exponents C_1 , C_2 and C_3 describe the dependences of the resistance function on f_y , A and E respectively. The values of the C_i parameters were derived for each considered test and FE simulation following the approach described in (Meng & Gardner, 2021), and the value of V_{rt} was then calculated using Equation (3.18).

$$N_{b,R} = C_0 f_y^{C_1} A^{C_2} E^{C_3} \quad (3.17)$$

$$V_{rt} = \sqrt{(C_1 V_{f_y})^2 + (C_2 V_A)^2 + (C_3 V_E)^2} \quad (3.18)$$

By comparing the experimental (or numerical) results $r_{e,i}$ with the design predictions $r_{t,i}$, the correction factor b and the COV of the errors V_δ were derived. Note that b was determined using Equation (3.19) instead of the least squares method specified in EN 1990 (2002), as recommended in (Meng et al., 2020). The considered test and FE data population was divided into subsets based on the steel grades and relative slenderness to avoid overestimating the scatter, and the values of b and V_δ were calculated based on these subsets. The design fractile factor $k_{d,n}$ was taken based on the total number of the data points in the original population, as recommended in EN 1990 (2002).

$$b = \frac{1}{n} \sum_{i=1}^n \frac{r_{c,i}}{r_{t,i}} \quad (3.19)$$

Finally, the required partial safety factors, denoted by γ_{MI}^* , were derived, and the results are summarised, along with the other key statistical results from the reliability analyses, in Tables 3.10 and 3.11 for the EC3 and proposed design approaches respectively. Note that the value of V_r varied with each considered test or FE data point, and the average values are shown in Tables 3.10 and 3.11. In general, the proposed design approach demonstrated a similar level of reliability to the existing EC3 approach, while improving the accuracy and consistency of the resistance predictions, as revealed previously in Section 3.5. Therefore, the proposed design rules are deemed suitable for inclusion into Eurocode 3 for the buckling design of welded homogeneous and hybrid I-section columns.

Table 3.10: Reliability analysis results for EC3 design approach

Column type	Buckling axis	Number of data points FE (test)	$k_{d,n}$	b	V_δ	V_r	γ_{MI}^*
Homogeneous welded I-section	Major	1500 (3)	3.097	1.061	0.070	0.054	1.061
	Minor	1500 (35)	3.097	1.088	0.104	0.058	1.059
Hybrid welded I-section	Major	1500 (2)	3.097	1.091	0.069	0.056	1.050
	Minor	1500 (0)	3.097	1.132	0.080	0.058	1.024

Table 3.11: Reliability analysis results for proposed design approach

Column type	Buckling axis	Number of data points FE (test)	$k_{d,n}$	b	V_δ	V_r	γ_{MI}^*
Homogeneous welded I-section	Major	1500 (3)	3.097	1.071	0.039	0.054	1.035
	Minor	1500 (35)	3.097	1.062	0.062	0.058	1.062
Hybrid welded I-section	Major	1500 (2)	3.097	1.075	0.047	0.055	1.051
	Minor	1500 (0)	3.097	1.077	0.050	0.055	1.054

3.7 Conclusions

The flexural buckling behaviour and design of both homogeneous and hybrid welded I-section columns of varying steel grades have been investigated in this chapter. The amplitudes and distributions of residual stresses in homogeneous and hybrid S235 to S960 steel welded I-sections were firstly analysed on the basis of existing experimental data collected from the literature, and a new residual stress model with different statistical characterizations (mean and characteristic) was proposed. An experimental programme was then carried out to generate underpinning test data on both homogeneous and hybrid welded I-section columns failing by in-plane flexural buckling about the major axis. The experimental programme comprised tensile coupon tests, initial global geometric imperfection measurements and five pin-ended major axis flexural buckling tests. Three different welded I-section profiles were investigated in the experimental programme – two homogeneous welded I-sections with different geometries made of S690 steel and one hybrid welded I-section with S690 steel flanges and an S355 steel web.

Following the experimental programme, a comprehensive numerical investigation was performed, in which FE models were initially developed and validated against the flexural buckling test results on welded I-section columns generated in the present study and collected from the literature, and then used to conduct parametric studies to broaden the range of available flexural buckling data on homogeneous and hybrid welded I-section columns. In total, 6000 numerical data – 3000 on homogeneous welded I-section columns and 3000 on hybrid welded I-section columns, were generated, covering a variety of steel grades, cross-section geometries and member slendernesses. Based on the established test and FE database, the accuracy and applicability of the relevant codified design rules, as specified in prEN 1993-1-1 (2019) and AISC 360 (2016), were assessed. It was shown that the current design provisions yield somewhat scattered buckling resistance predictions. A new design approach based on the current EC3 design rules was then proposed, where the influence of the yield strength on the flexural buckling resistances of homogeneous and hybrid welded I-section columns was more rationally captured by using a yield strength-dependent imperfection factor. The proposed method was shown to offer a higher degree of accuracy and consistency in the predictions of flexural buckling resistances for both homogeneous and hybrid welded I-section columns with

varying steel grades. Reliability analysis was finally conducted for the proposed design method in accordance with EN 1990 (2002), indicating that the current EC3 partial safety factor of unity can be safely employed with the proposed method for the design of both homogeneous and hybrid welded I-section columns failing by in-plane flexural buckling. Further research on the flexural buckling behaviour and design of Class 4 (slender) welded I-section columns, which may experience local-global interactive buckling, as well as the flexural-torsional buckling behaviour of HSS homogeneous and hybrid welded I-section beams are recommended.

Chapter 4

Welded I-section beams

4.1 Introduction

This chapter features experimental, numerical and design-oriented studies into HSS beams. A total of six in-plane three-point bending tests on HSS homogeneous and hybrid welded I-sections are first described in Section 4.2. Three different I-sections – two homogeneous S690 steel welded I-sections and one hybrid welded I-section with S690 steel flanges and an S355 steel web, were adopted in the experimental programme. The beam specimens were bent about the major axis and a bespoke restraint rig was employed to prevent out-of-plane instability. Following the experimental study, finite element (FE) models were established and validated against the test results obtained from the present study as well as those collected from the literature (Sun et al., 2019), as described in Section 4.3. The validated FE models were used to perform a series of parametric studies to generate supplementary numerical data on HSS homogeneous and hybrid welded I-section beams, covering a wide range of cross-section aspect ratios, cross-section slendernesses, loading configurations and steel grades. Based on the obtained test and FE results, the rotation capacity of HSS homogeneous and hybrid welded I-section beams was investigated and the suitability of the current Eurocode slenderness limits (EN 1993-1-1, 2019) was assessed.

4.2 Experimental programme

4.2.1 General

An experimental study was performed to investigate the major-axis flexural behaviour and rotation capacity of HSS homogeneous and hybrid welded I-section beams subjected to mid-span concentrated load. Specifically, a total of six three-point major axis bending tests, with two on each of the three investigated welded I-section profiles (i.e. HSS-I-65×116×8×8, HSS-I-80×136×8×8 and HYB-I-80×136×8×8), were conducted. Three-point bending tests were chosen to investigate the bending resistance and rotation capacity of the studied cross-sections under a moment gradient; this is a common scenario in plastic designed frames. Each beam specimen was labelled by a unique identifier, e.g. HSS-I-65×116×8×8-3PB, where HSS-I (or HYB-I) denotes a homogeneous HSS welded I-section made of S690 steel for both its flanges and web (or a hybrid welded I-section with its flanges made of S690 steel and web made of S355 steel), 65×116×8×8 are the nominal dimensions of the welded I-section in millimetres (i.e. flange width $B \times$ outer section height $H \times$ flange thickness $t_f \times$ web thickness t_w) as illustrated in Figure 4.1, and 3PB represents three-point bending tests. For repeated tests, the letter “R” is added as a suffix to the specimen identifier. All three welded I-section sizes were fabricated from 8 mm-thick quenched and tempered S690 steel plates and hot-rolled S355 steel plates by means of gas metal arc welding with a nominal weld leg length (t_{weld}) of 5.6 mm (i.e. a weld throat thickness of 4 mm), as shown in Figure 4.1.

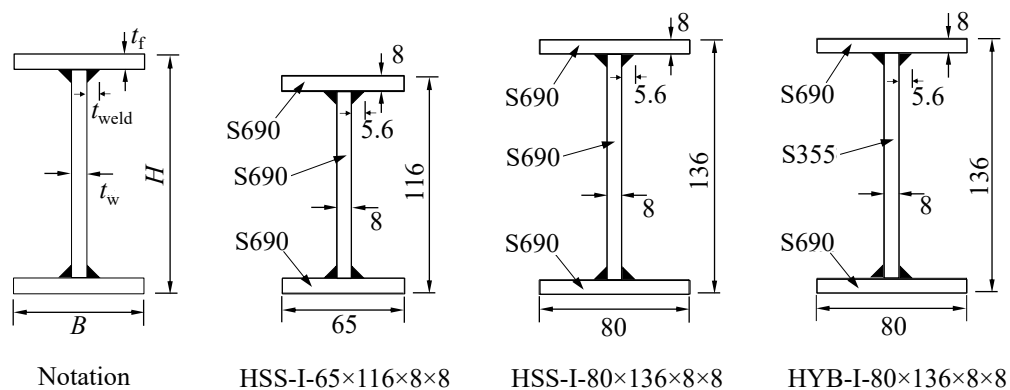


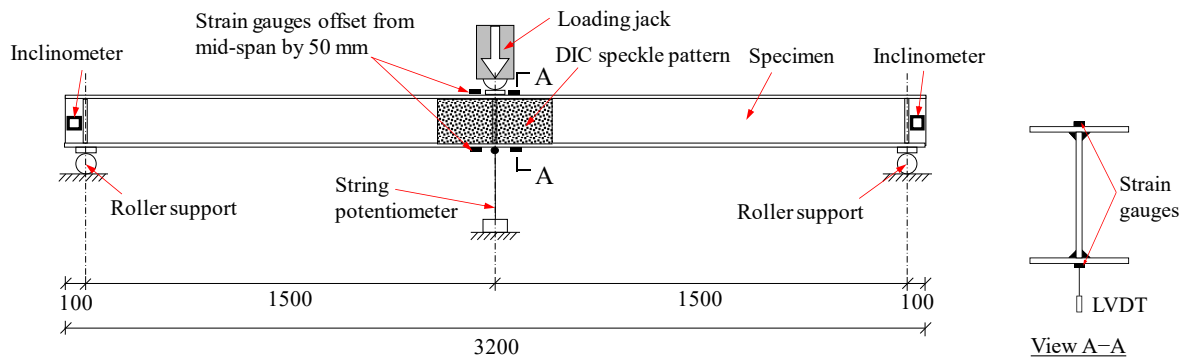
Figure 4.1: Notation and nominal dimensions of the three investigated welded I-sections (dimensions in mm).

4.2.2 Geometry measurements

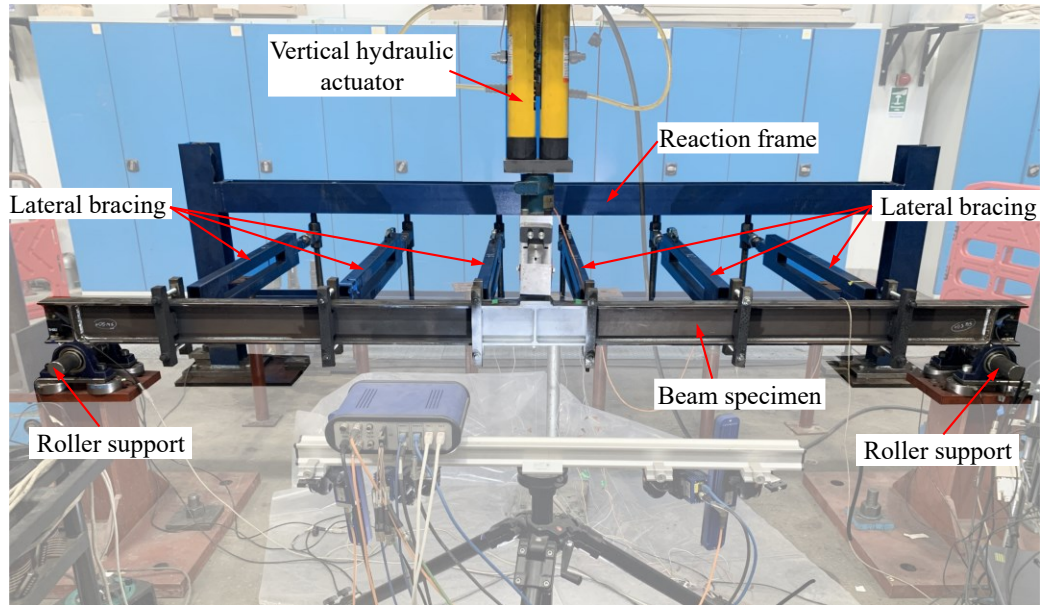
The tensile coupon tests on all the investigated material and the obtained test results have been presented in Section 3.3.2. The nominal length of all beam specimens L was 3200 mm, with an overhang of 100 mm at each end beyond the corresponding roller support, resulting in a span of $L_{\text{span}} = 3000$ mm for each tested specimen, as illustrated in Figure 4.2(a). The average measured geometrical dimensions for each beam specimen are reported in Table 4.1, together with the cross-sectional properties calculated based on the measured dimensions, where A is the cross-sectional area, I is the second moment area about the major axis, W_{el} and W_{pl} are the elastic and plastic section moduli about the major axis, respectively, and M_{el} and M_{pl} are the elastic and plastic bending moment capacities about the major axis, respectively. Note that, in Table 4.1, M_{el} for the hybrid welded I-sections is defined as the bending moment at which the stress at the extreme outer-fibre of the flange reaches its yield strength, as detailed further in Section 4.4.6. The three investigated welded I-sections are all Class 1 cross-sections under pure bending according to the slenderness limits specified in prEN 1993-1-1 (2019).

Table 4.1: Average measured geometrical dimensions and cross-sectional properties of beam specimens

Beam specimen	L mm	L_{span} mm	B mm	H mm	t_f mm	t_w mm	A mm ²	I mm ⁴	W_{el} mm ³	W_{pl} mm ³	M_{el} kNm	M_{pl} kNm
HSS-I-65×116×8×8-3PB	3200	3000	64.42	116.3	8.40	8.45	1985.8	3.995×10^6	6.869	7.112	54.03	64.74
HSS-I-65×116×8×8-3PB-R	3200	3000	62.22	116.5	8.40	8.42	1947.5	3.900×10^6	6.695	6.938	52.66	63.26
HSS-I-80×136×8×8-3PB	3200	3000	79.59	136.6	8.41	8.43	2411.8	6.934×10^6	1.015	1.044	79.81	94.20
HSS-I-80×136×8×8-3PB-R	3200	3000	79.42	136.7	8.44	8.44	2415.0	6.949×10^6	1.016	1.046	79.93	94.36
HYB-I-80×136×8×8-3PB	3200	3000	79.71	136.7	8.40	8.08	2370.8	6.893×10^6	1.008	1.036	75.53	80.99
HYB-I-80×136×8×8-3PB-R	3200	3000	79.61	136.6	8.37	8.18	2376.3	6.877×10^6	1.006	1.035	75.34	80.79



(a) Schematic diagram (dimensions in mm)



(b) Lateral restraint system

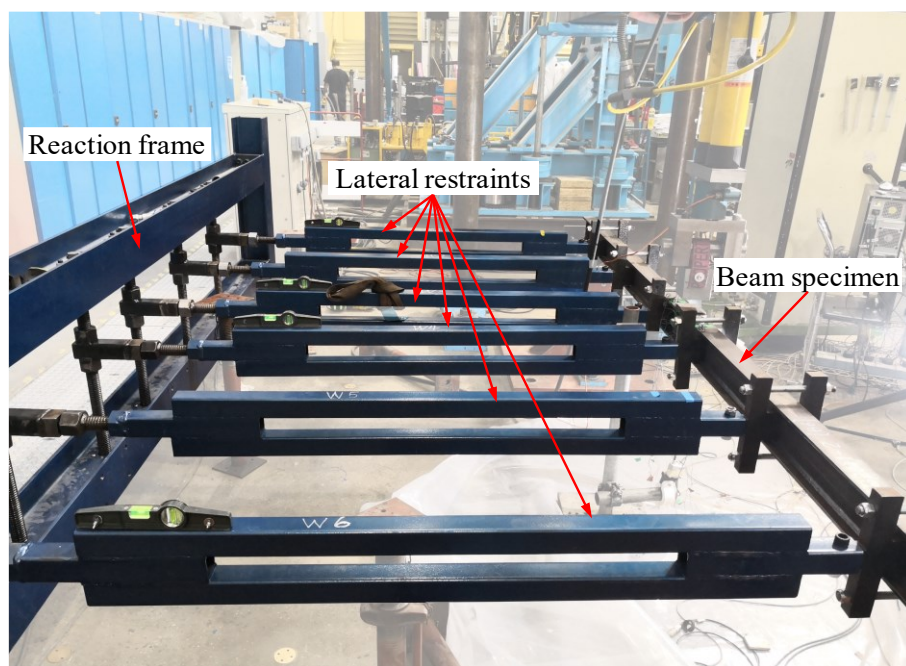
Figure 4.2: Configuration of beam specimens and experimental setup for major axis three-point bending tests.

4.2.3 Three-point bending tests

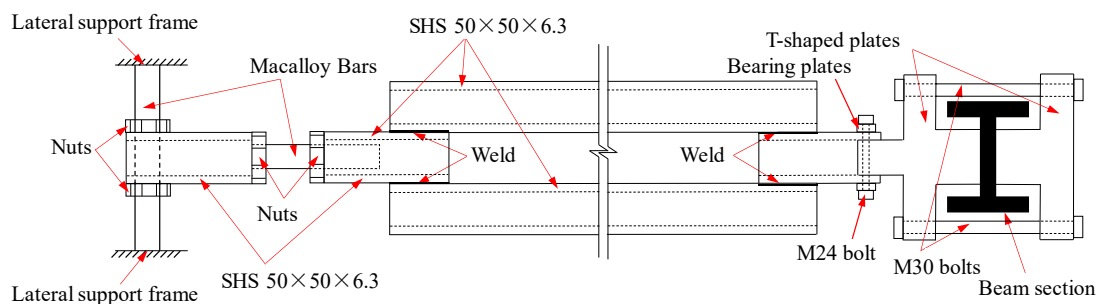
A schematic view of the arrangement of the test rig and the instrumentation for the three-point in-plane bending tests is shown in Figure 4.2(a). Each beam specimen was simply supported between a pair of steel rollers, which permitted the beam ends to undergo rotations about the axis of bending as well as longitudinal displacements. The tested beam specimens were loaded at mid-span using an Instron 250 kN testing machine under displacement control at a vertical displacement rate of 2.0 mm/min. Stiffeners and bearing plates were welded at the mid-span and the ends of the beam specimens to prevent web crippling under concentrated transverse loading.

A bespoke lateral restraint system was designed and employed to prevent lateral and torsional deformations of the beam specimens and hence to prevent lateral-torsional buckling (LTB). The lateral restraint system was composed of a reaction frame and six lateral restraints, as shown in Figure 4.2(b). Each lateral restraint comprised two T-shaped plates which were used to clamp the section of the beam specimen by means of two M30 bolts, and a bracing beam made of two parallel S460 square hollow section (SHS) members, as illustrated in Figure 4.3. The assembled T-shaped plates were bolted to the bracing beam with an M24 bolt, the shank

of which was greased to facilitate free rotation. Each lateral restraint was connected to the reaction frame via two 35 mm-diameter Macalloy bars as shown in Figure 4.3, enabling the lateral restraint to be adjustable in both length and height. During testing, a ratchet spanner was employed to rotate the threaded bars installed in the reaction frame in order to adjust the height of the lateral restraints to match the corresponding vertical deflections of the beam specimens. Similar lateral restraint systems have also been successfully employed in the columns tests reported in Chapter 3 and the frame tests presented later in Chapter 6.



(a) Photo



(b) Schematic drawing

Figure 4.3: Lateral restraint system for three-point bending tests

Four strain gauges were affixed to the top and bottom flanges of the beam sections, at a distance of 50 mm from the stiffened mid-span section, as shown in Figure 4.2(a), to monitor the strain development histories during testing. A string potentiometer was attached to the bottom flange of the beam at mid-span to record the vertical deformation of the beam during loading. Two inclinometers were mounted onto the webs of the extended overhangs to record the end rotations about the bending axis. The test outputs, including the applied vertical load and readings from the strain gauges, string potentiometer and inclinometers were recorded at 2-second intervals using the data acquisition equipment DATASCAN.

To gain further insight into the strain development at the mid-span region of the beam specimens where inelastic local buckling occurred, in addition to the traditional instrumentation described above, a two-camera digital image correlation (DIC) system was employed. The DIC setup is shown in Figure 4.4, where two cameras, with the focal lengths set to 35 mm, were utilised for the image acquisition. Prior to testing, the mid-span region of all beam specimens was painted white and then sprayed with a random black speckle pattern, as shown in Figure 4.4. Images were taken at 2-second intervals and processed using the DaVis version 8.4.0 imaging software (Davis, 2017).

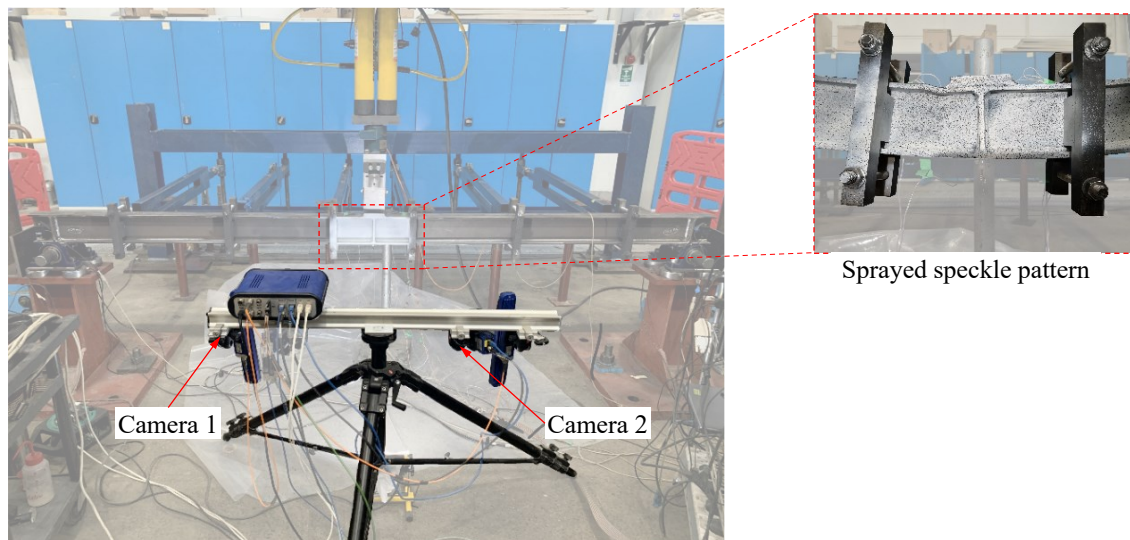


Figure 4.4: DIC setup and speckle pattern for three-point bending tests.

All the tested beam specimens failed by inelastic local buckling in the maximum moment region adjacent to the stiffened mid-span, a typical example of which is shown in Figure 4.5. The normalised mid-span bending moment (M)-mid-span rotation (φ) curves for all tested beam specimens are plotted in Figure 4.6, where the mid-span bending moment M was calculated from the applied vertical load, with allowance for the horizontal movement at the supports (Meng & Gardner, 2020b), and the mid-span rotation φ was determined as the sum of the end rotations recorded by the two inclinometers at the overhangs, as shown in Figure 4.2(a). The key test results are summarised in Table 4.2, where M_u is the ultimate bending moment at mid-span, φ_{pl} is the elastic component of the mid-span rotation when M_{pl} is first attained, determined by Equation (4.1), φ_u is the mid-span rotation at M_u and R is the rotation capacity. The rotation capacity R is defined by Equation (4.2), where φ_{rot} is the mid-span rotation when the moment at mid-span falls below M_{pl} on the descending branch. Note that the maximum recorded rotation φ_{max} , instead of φ_{rot} , was employed in Equation (4.2) to calculate the rotation capacities of the presented three-point bending tests since these tests exceeded their peak moments, but were discontinued prior to the bending moment dropping below M_{pl} ; the obtained rotation capacities, as listed in Table 4.2 may therefore be considered to be lower bounds.

$$\varphi_{pl} = \frac{M_{pl} L_{span}}{2EI} \quad (4.1)$$

$$R = \frac{\varphi_{rot}}{\varphi_{pl}} - 1 \quad (4.2)$$

With regards to beams subjected to four-point bending, the results of which have been collected from the literature and generated in the FE parametric study described later, the curvature in the constant moment region was utilised for calculating the rotation capacity, as given by Equation (4.3), where κ_{pl} is the elastic curvature corresponding to the plastic bending moment M_{pl} and κ_{rot} is the curvature when the mid-span bending moment falls below M_{pl} on the descending branch. The curvature is defined based on the assumption that the deformed shape of the constant moment region (i.e. the mid-region between two loading points) represents a segment of a circular arc (of radius r), resulting in the expression of Equation (4.4), where D_M is the vertical displacement at mid-span and D_L is the average vertical displacement at the two

loading points. This definition of rotation capacity for beams in four-point bending has been employed in many previous studies (Meng & Gardner, 2020b; Wang et al., 2016c).

$$R = \frac{\kappa_{\text{rot}}}{\kappa_{\text{pl}}} - 1 \quad (4.3)$$

$$\kappa = \frac{8(D_M - D_L)}{4(D_M - D_L)^2 + L_M^2} \quad (4.4)$$

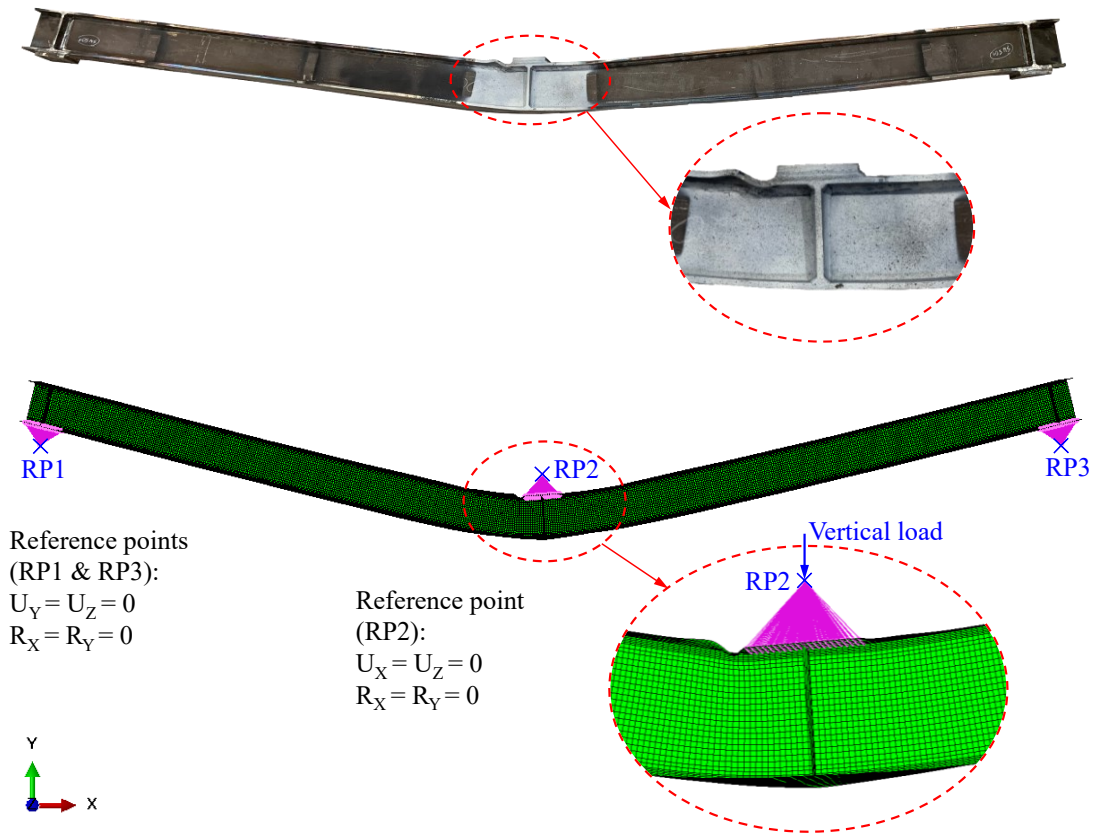
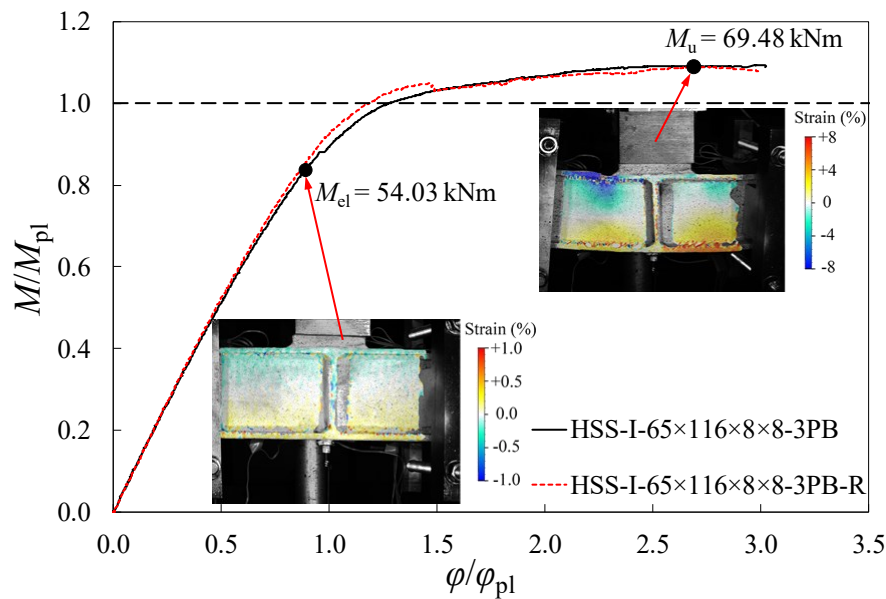


Figure 4.5: Comparison between test and FE failure modes for HYB-I-80×136×8×8-3PB.

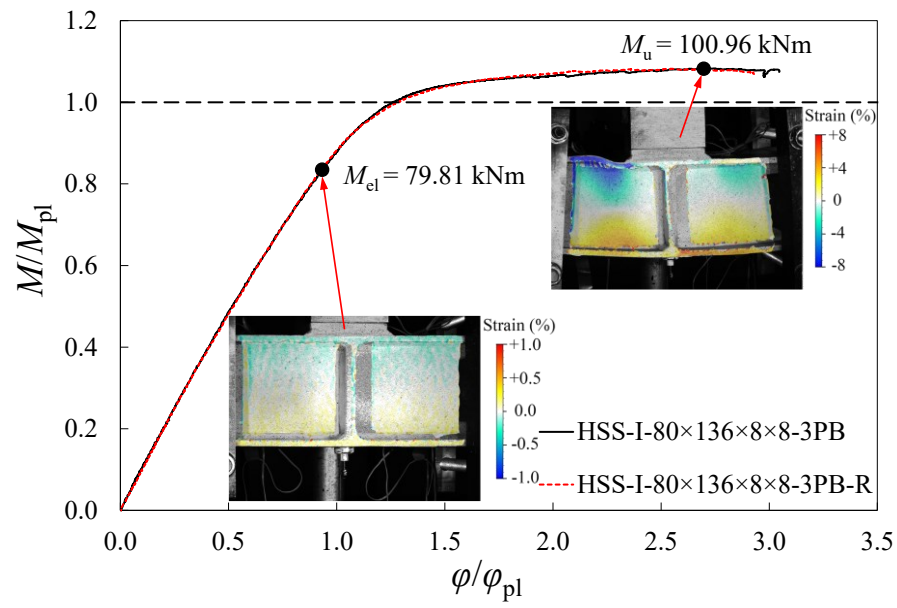
Table 4.2: Key experimental results from three-point bending tests.

Specimen	M_u kNm	M_u/M_{pl} -	ϕ_{pl} rad	ϕ_u rad	R -
HSS-I-65×116×8×8-3PB	69.48	1.05	0.11	0.35	>2.14
HSS-I-65×116×8×8-3PB-R	67.50	1.05	0.11	0.35	>2.23
HSS-I-80×136×8×8-3PB	100.96	1.05	0.10	0.30	>1.93
HSS-I-80×136×8×8-3PB-R	100.91	1.05	0.11	0.28	>1.67
HYB-I-80×136×8×8-3PB	91.36	1.12	0.09	0.30	>2.50
HYB-I-80×136×8×8-3PB-R	90.78	1.11	0.08	0.32	>2.91

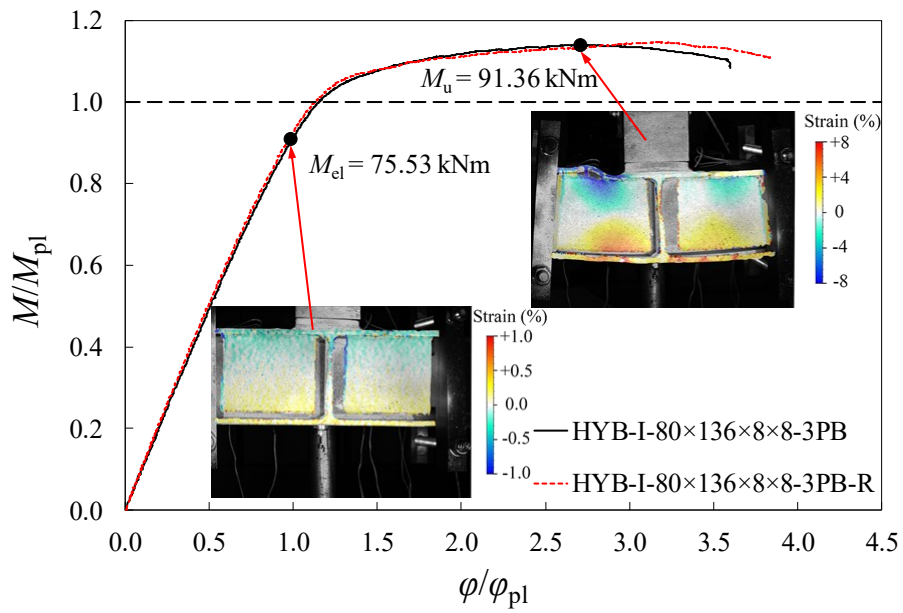
The accuracy of the measurements obtained from the DIC system has been confirmed by comparing the displacement and strain histories obtained from the DIC system to those measured from the string potentiometers and strain gauges, respectively, at the corresponding location of each beam specimen. A more detailed description of the DIC validation has been reported in Chapter 6. The longitudinal strain fields within the mid-span regions of beam specimens HSS-I-65×116×8×8-3PB, HSS-I-80×136×8×8-3PB and HYB-I-80×136×8×8-3PB, obtained from the DIC analyses at two representative load levels: (1) the load at which the mid-span bending moment reached the elastic bending moment M_{el} , and (2) the maximum applied load corresponding to M_u , are displayed in Figure 4.6, where the development of the inelastic local buckling can be observed. Note that the positive and negative strains in Figure 4.6 represent tensile strains and compressive strains, respectively.



(a) HSS-I-65×116×8×8-3PB and HSS-I-65×116×8×8-3PB-R



(b) HSS-I-80x136x8x8-3PB and HSS-I-80x136x8x8-3PB-R



(c) HYB-I-80x136x8x8-3PB and HYB-I-80x136x8x8-3PB-R

Figure 4.6: Normalised mid-span bending moment-mid-span rotation curves and longitudinal strain fields at different representative loads obtained from DIC for three-point bending specimens.

4.3 Numerical simulations

4.3.1 General

In parallel with the experimental studies, comprehensive numerical simulations were performed to investigate the in-plane flexural behaviour of HSS homogeneous and hybrid welded I-section beams using the finite element (FE) analysis software ABAQUS (2018). The basic modelling assumptions and validation of the FE models are described in Section 4.3.2 and Section 4.3.3, respectively, while parametric studies, which were carried out to generate further numerical data considering different key parameters, are presented in Section 4.3.4.

4.3.2 Basic modelling assumptions

The four-noded shell element S4R with reduced integration and finite membrane strains was used for all simulations in the present study; this element has been shown to be capable of modelling the buckling behaviour of plated structural elements and has been successfully employed in several previous studies on I-sections subjected to different loading conditions (Bu & Gardner, 2018, 2019a; Yun et al., 2018b). The FE models were validated against the three-point bending tests performed in the present study, as well as the four-point bending tests reported in (Sun et al., 2019). The measured stress-strain curves transformed into true stress-logarithmic plastic strain curves for incorporation into ABAQUS (2018), were used in both cases.

The measured average cross-sectional dimensions of the welded I-sections of each beam specimen were carefully modelled using the technique introduced in Section 3.4.2. The nominal weld leg length of 5.6 mm was employed in the validation FE models. In the transverse direction, a mesh size of approximately $(B+H)/40$ was adopted in the flange and web plates; the same mesh size was also assigned uniformly in the longitudinal direction of the modelled beam specimens, such that the element aspect ratio was close to unity. The mesh size was selected based on a prior mech sensitivity study, where mesh sizes ranging between $(B+H)/100$ and $(B+H)/20$ were examined. A mesh size of $(B+H)/40$ which provided accurate results at reasonable computational cost, was finally selected to discretize the FE models.

Residual stresses were introduced into the FE models using the novel residual stress model proposed in Section 3.2. As recommended, the proposed residual stress pattern with the mean value of the maximum tensile residual stress (i.e. Equation (3.1)) is used for the validation of FE models (to match test results), while the proposed residual pattern with the upper characteristic value of the maximum tensile residual stress (i.e. Equation (3.2)) is employed for parametric studies (to provide safe-sided capacity data). The membrane residual stresses determined from the residual stress pattern were incorporated into the developed FE models as an initial stress condition using the ABAQUS command `*INITIAL CONDITIONS`.

Initial local and global geometric imperfections were introduced into the developed FE models by modifying the nodal coordinates of the perfect geometry; the modelling assumptions and techniques have been described in Section 3.4.2. Note that the use of tolerance-based imperfection amplitudes would be expected to yield slightly conservative resistance predictions in comparison to experimental results.

Suitable boundary and loading conditions were employed in the FE models to mimic the corresponding test setup, as described in Section 4.2.3 for the three-point bending tests and reported in (Sun et al., 2019) for the four-point bending tests. All degrees of freedom of the upper (i.e. for the loading points) or lower (i.e. for the support points) flange nodes over a region corresponding to the location of the bearing plates (or wooden blocks (Sun et al., 2019)) employed in the tests were coupled to a corresponding reference point, where the boundary and loading conditions were assigned, as detailed in Figure 4.5. Stiffeners were also modelled at the loading and support sections where concentrated loads were applied in order to prevent localised web crippling failure, as employed in the tests. The stiffeners were tied to their corresponding cross-sections using `*TIE` constraints, and their material properties were assumed to be the same as those of the corresponding flange plates. The out-of-plane displacement degree of freedom of the nodes located at the cross-sections where lateral restraints were provided was also constrained to prevent lateral-torsional buckling. The modified Riks method was employed for the geometrically and materially nonlinear analyses (GMNIA), enabling the post-ultimate response of the modelled specimens to be tracked.

4.3.3 Validation of FE models

The developed FE models were validated against the results from the three-point bending tests described in Section 4.2.3 and the four-point bending tests reported in (Sun et al., 2019) in terms of ultimate bending moments, full moment-deformation histories and failure modes. The ultimate bending moments derived from the FE models $M_{u,FE}$ are plotted against the results from the three-point and four-point bending tests $M_{u,test}$ in Figure 4.7, indicating good agreement, with the mean value of the ratio of $M_{u,test}/M_{u,FE}$ being 0.98 and the corresponding coefficient of variation being 0.02. The experimental moment-deformation curves and failure modes were also well replicated by the FE models, as illustrated by the typical examples displayed in Figure 4.8 and Figure 4.5, respectively. It can thus be concluded that the established FE models are capable of replicating the flexural behaviour of the homogeneous and hybrid HSS welded I-section beams subjected to major axis bending.

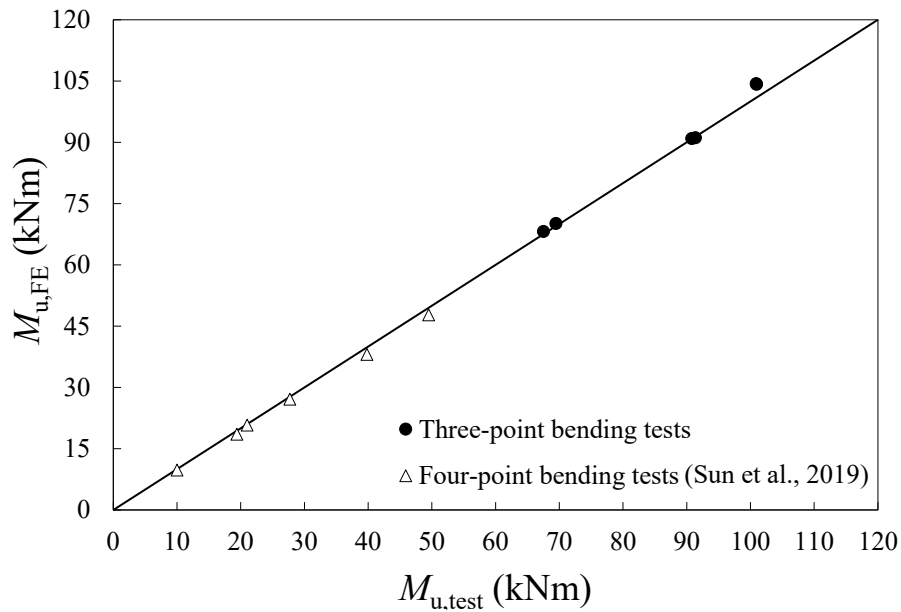
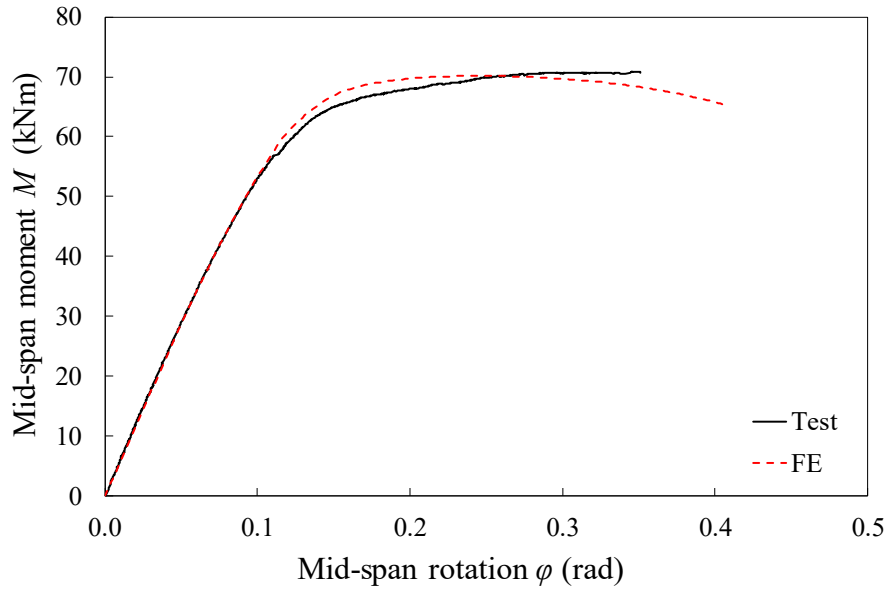
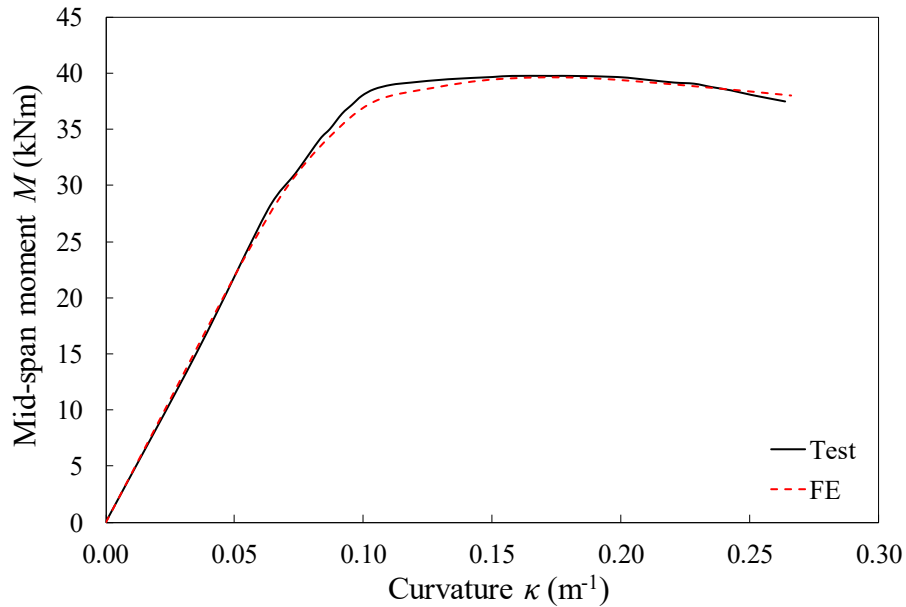


Figure 4.7: Comparison between ultimate bending moments obtained from FE models $M_{u,FE}$ and tests $M_{u,test}$.



(a) Three-point bending test carried out in the present study (HSS-I-65×116×8×8-3PB)



(b) Four-point bending test carried out by Sun et al. (I-100×100×5-MA) (Sun et al., 2019)

Figure 4.8: Typical comparisons between moment-deformation curves obtained from tests and FE models.

4.3.4 Parametric studies

Upon the validation of the developed FE models, parametric studies were conducted with the aim of expanding the experimental data pool for homogeneous and hybrid welded I-section beams over a wide range of cross-section aspect ratios, cross-section slendernesses, steel grades and loading configurations. Regarding the geometry of the modelled I-sections, the flange

width B was kept constant at 100 mm, while four outer section heights H of 100, 150, 200 and 300 mm were considered, resulting in cross-section aspect ratios ranging from 1.0 to 3.0. The web thickness t_w of each modelled I-section was set to be 0.6 of the corresponding flange thickness t_f (i.e. $t_w = 0.6t_f$), which represents a typical web-to-flange thickness ratio for standard hot-rolled UB and IPE profiles. A total of 20 flange thicknesses t_f , varying from 3.4 mm to 17 mm, were considered for each aspect ratio in order to achieve a broad spectrum of cross-section geometries and flange slendernesses that cover all four classes (i.e. Class 1-4) according to the slenderness limits set out in prEN 1993-1-1 (2019). The weld leg length t_{weld} of all the modelled welded I-sections was set equal to 5.6 mm. Three steel grades (S355, S460 and S690) were considered in the parametric studies, allowing the following five combinations of flanges/webs to be studied: (1) homogeneous S690 steel welded I-sections; (2) homogeneous S460 steel welded I-sections; (3) homogeneous S355 steel welded I-sections; (4) hybrid welded I-sections with S690 steel flanges and S460 steel webs; and (5) hybrid welded I-sections with S690 steel flanges and S355 steel webs. The bilinear plus nonlinear hardening material model developed by Yun and Gardner (2017) was utilised to represent the full-range stress-strain curves of the S355, S460 and S690 steels. The basic material properties (i.e. E , f_y and f_u) for the S355, S460 and S690 steels used in the parametric studies were taken as the nominal values given in prEN 1993-1-1 (2019), as listed in Table 4.3, while the full-range stress-strain curves – see Figure 4.9, as well as the other key material properties such as ϵ_u and ϵ_{sh} , were derived using the predictive model set out in Yun and Gardner (2017). It should be noted that the structural responses of FE models with predicted stress-strain curves (Yun & Gardner, 2017), based on measured yield and tensile strengths (see Table 3.4), and full measured stress-strain curves, were very similar.

Table 4.3: Summary of key material properties for S355, S460 and S690 steels employed in parametric studies.

Steel grade	E	f_y	f_u	f_u/f_y	ϵ_u	ϵ_{sh}
	N/mm ²	N/mm ²	N/mm ²	-	-	-
S355	210000	355	490	1.38	0.17	0.017
S460	210000	460	540	1.17	0.09	0.030
S690	210000	690	770	1.12	0.06	0.030

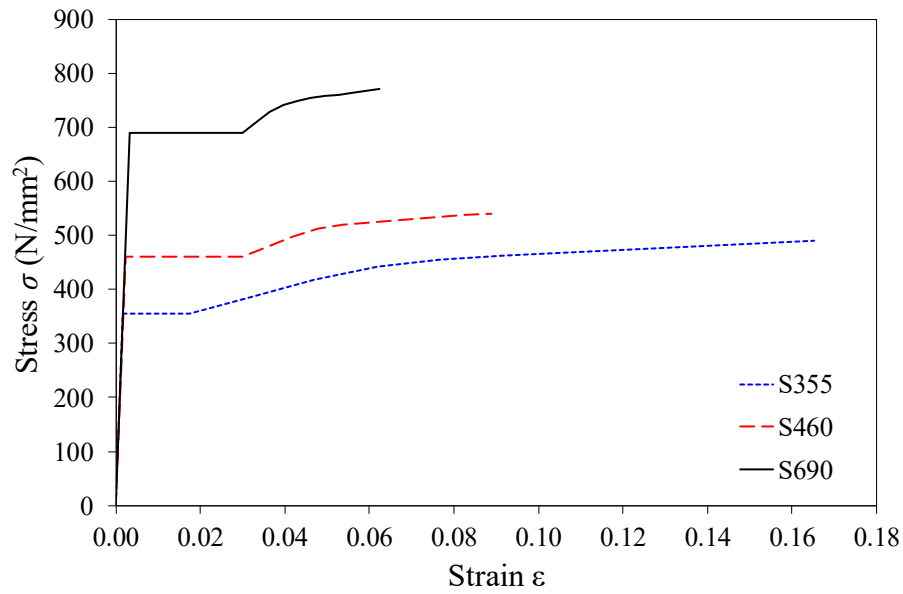
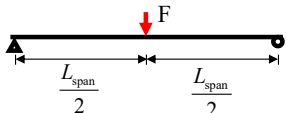
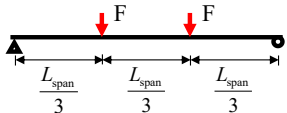


Figure 4.9: Full-range stress-strain curves for S355, S460 and S690 steels employed in parametric studies (Yun & Gardner, 2017)

Table 4.4: Ranges of variation of parameters considered in parametric studies.

Loading configuration	Material combination (Flange / Web)	B	H	t_f	t_w	t_{weld}	L_{span}
		mm	mm	mm	mm	mm	mm
 Three-point bending	S690 / S690	100	100/150/ 200/300	3.4-17	$0.6t_f$	5.6	$15H$
	S460 / S460						
	S355 / S355						
	S690 / S355						
	S690 / S355						
 Four-point bending	S690 / S690	100	100/150/ 200/300	3.4-17	$0.6t_f$	5.6	$15H$
	S460 / S460						
	S355 / S355						
	S690 / S355						
	S690 / S355						

The span of each beam model was set at 15 times its outer section height H , and each beam model was loaded in both three-point bending and four-point bending. In order to prevent lateral torsional buckling, uniform and symmetrical lateral restraints were provided along the length of the beams at intervals approximately equal to the length that corresponds to a non-dimensional lateral torsional buckling slenderness $\bar{\lambda}_{LT}$ value of 0.2 under pure bending. The lateral restraints were applied at the top and bottom web-to-flange junctions of the sections in order to avoid any influence on the local buckling behaviour of the beam specimens. The ranges

of variation of parameters considered in the parametric studies are listed in Table 4.4, leading to the simulation of a total of 800 welded I-section beams; the generated FE results are presented and analysed in the following section.

4.4 Analysis of results and design recommendations for HSS homogeneous welded I-section beams

4.4.1 General

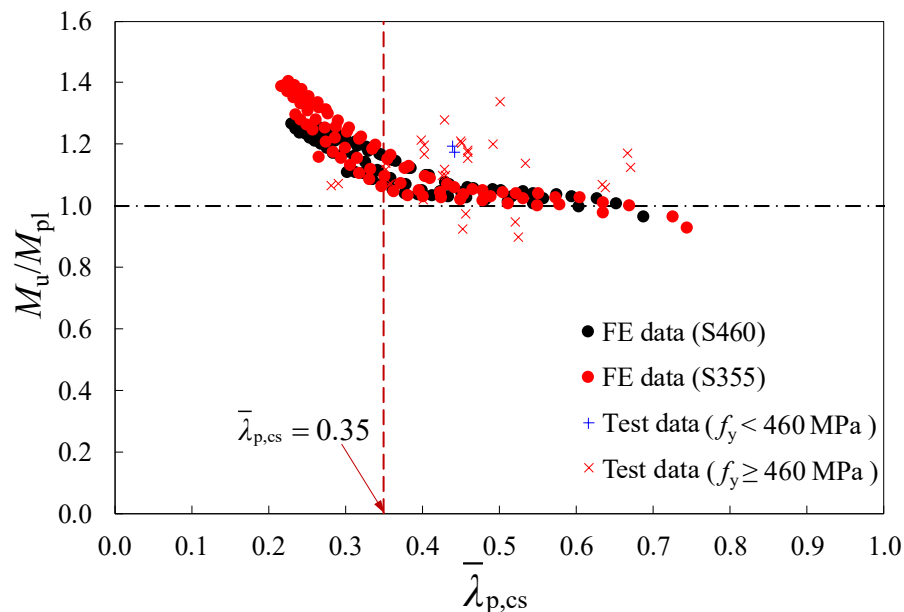
Current structural steel design standards use the concept of cross-section classification to consider the influence of local buckling on the behaviour and design of cross-sections. Four discrete behavioural classes (i.e. Class 1-4) are typically defined (EN 1993-1-1, 2019). A Class 1 cross-section can develop its plastic moment capacity M_{pl} in bending and is deemed to have sufficient rotation capacity to allow full redistribution of moments. A Class 2 cross-section is also able to reach its plastic moment capacity M_{pl} but with lower rotation capacity in bending. For a Class 3 cross-section subjected to bending, local buckling occurs after attainment of the elastic moment M_{el} , but before the plastic moment M_{pl} is reached. Hence, the bending resistance of a Class 3 cross-section is limited to its elastic moment capacity M_{el} , or to a linearly interpolated moment between M_{el} and M_{pl} (EN 1993-1-1, 2019). Finally, for a Class 4 cross-section in bending, local buckling occurs prior to the elastic moment being reached; this is accounted for through the effective width method. The classification of a plate element within the cross-section is based on its width-to-thickness ratio (b_f/t_f or h_w/t_w), material (through $\varepsilon = (235/f_y)^{0.5}$), boundary conditions and stress distribution. The overall class of a cross-section is defined as the class of the most slender plate element in compression.

Existing experimental results on HSS welded I-section beams have been collected from the literature and are summarised in Table 2.1 and Table 2.2. These test results, together with the numerical data generated from the parametric studies, are used in Sections 4.4.2 to 4.4.4 to assess the influence of material grade, cross-section aspect ratio and loading configuration on the ultimate bending resistances and rotation capacities of welded I-section beams subjected to major axis bending. The accuracy of the current cross-section slenderness limits for outstand

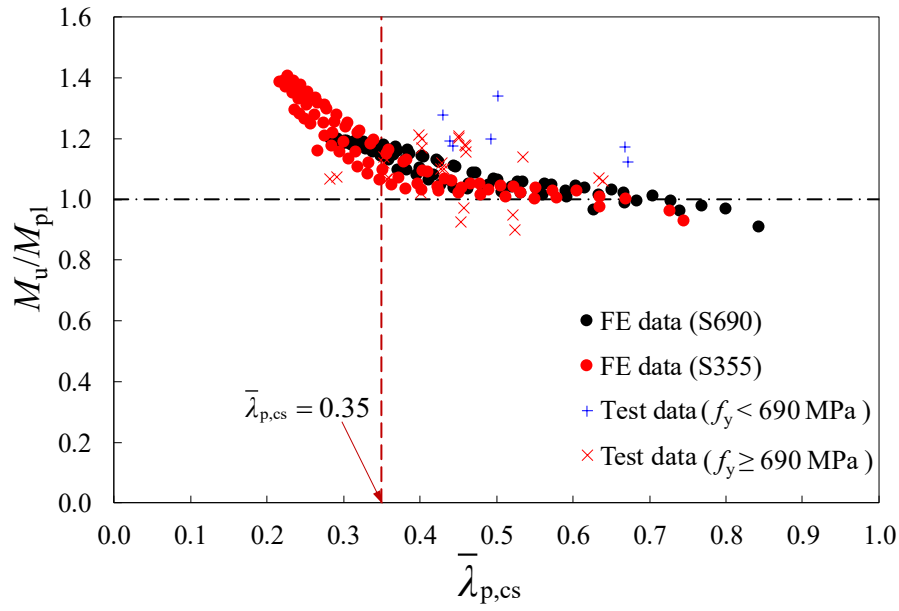
plate elements in compression and internal plate elements in bending, as specified in Eurocode 3 (EN 1993-1-1, 2019; EN 1993-1-12, 2007), are then evaluated in Section 4.4.5. Finally, design recommendations for hybrid welded I-section beams are considered in Section 4.4.6.

4.4.2 Influence of material grade

In Figure 4.10, the ultimate bending moments M_u obtained from the three-point bending tests and numerical investigations in the three-point bending configuration are normalised by the cross-section plastic bending moment M_{pl} , and plotted against the normalised cross-section slenderness $\bar{\lambda}_{p,cs} = (f_y / \sigma_{cr})^{0.5}$, where σ_{cr} is the elastic local buckling stress of the full cross-section, determined using the finite strip software CUFSM (Schafer & Adany, 2006); simplified analytical expressions are also available for determining the elastic local buckling stress of full structural cross-sections (Gardner et al., 2019), considering plate element interaction effects. The normalised ultimate bending moments M_u/M_{pl} of the S690 steel beams and S460 steel beams in three-point bending are compared with those of the S355 steel beams in Figure 4.10(a) and (b) respectively. The rotation capacities of the beams are plotted in a similar manner in Figure 4.11. The test data points are slightly more scattered than the FE simulations, as addressed in Section 4.4.5.2.

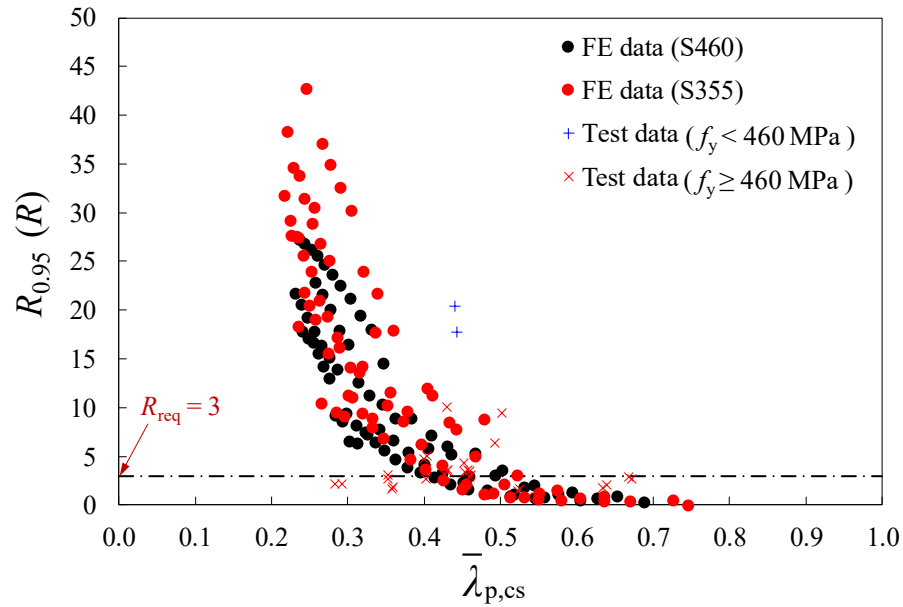


(a) Comparison of normalised ultimate bending capacity (M_u/M_{pl}) of S460 and S355 steel beams in three-point bending

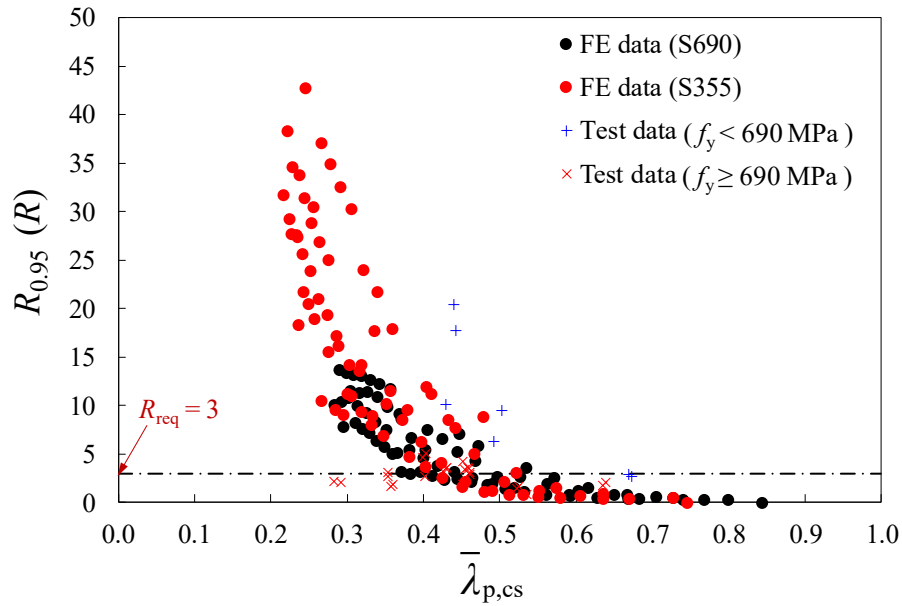


(b) Comparison of normalised ultimate bending capacity (M_u/M_{pl}) of S690 and S355 steel beams in three-point bending

Figure 4.10: Influence of material grade on normalised ultimate bending moment M_u/M_{pl} of welded I-section beams in three-point bending.



(a) Comparison of rotation capacity of S460 and S355 steel beams in three-point bending



(b) Comparison of rotation capacity of S690 and S355 steel beams in three-point bending
Figure 4.11: Influence of material grade on rotation capacity of welded I-section beams in three-point bending.

It can be observed from Figure 4.10 and Figure 4.11 that the data points for the HSS steel beams (both S690 and S460 steels) generally follow the same trend as those of the S355 steel beams, especially for beams with slender cross-sections, the failure of which is generally triggered by local buckling at strains below ε_{sh} (i.e. strain hardening is not experienced). Slightly lower normalised ultimate bending moment capacities were achieved for the stocky ($\bar{\lambda}_{p,cs} \lesssim 0.35$) S460 steel beams, compared with the corresponding stocky S355 steel beams. The stocky S690 steel beams were able to reach their plastic bending moment M_{pl} , but with the lowest normalised ultimate bending moment capacities. Similarly, the HSS beams exhibited somewhat lower rotation capacities compared with those of the S355 steel beams, as depicted in Figure 4.11. The decrease in the normalised ultimate bending moment capacities M_u/M_{pl} and the rotation capacity $R_{0.95} (R)$ for HSS beams with stocky cross-sections relates to their lower level of strain hardening, as indicated in Table 4.3. Note that throughout this section, rotation capacities have been determined from the numerical simulations using a reduced bending moment $0.95M_{pl}$ instead of the plastic bending moment M_{pl} and denoted $R_{0.95}$ (instead of R), as depicted in Figure 4.12; the purpose of this is to provide a more stable measure of rotation capacity and avoid anomalies in the results, since the moment-rotation (curvature) curves for stocky beams are typically characterised by a long plateau (particularly in four-point bending)

when the cross-section bending moment is in the region of the full plastic bending moment M_{pl} before descending. It is the length of the plateau region that dictates the ability of a cross-section to redistribute moments (rather than the specific value of moment attained), and hence this approach is deemed to be appropriate. The same approach has been employed previously by (Bartsch et al., 2021; Chan & Gardner, 2008; Lay & Galambos, 1965; European Commission et al., 1998).

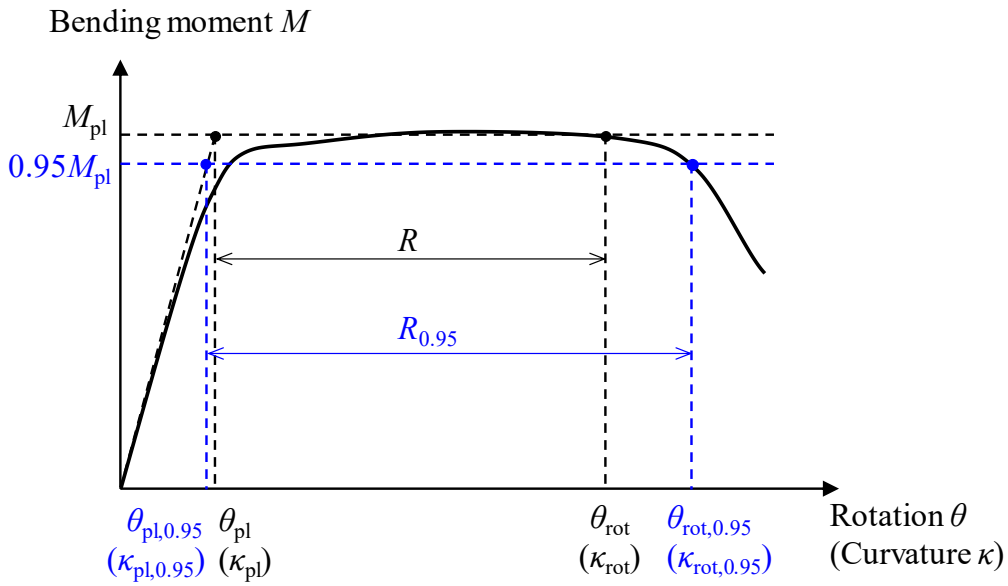
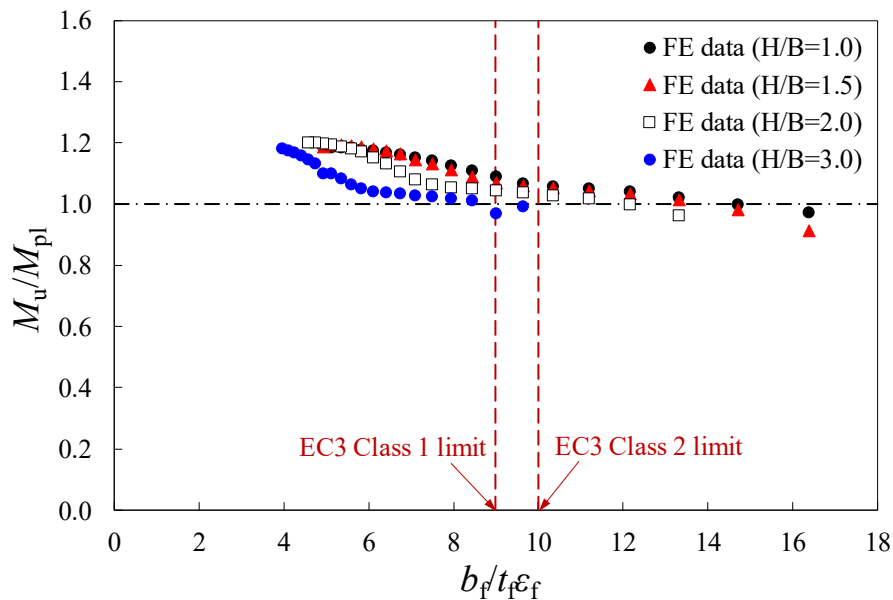


Figure 4.12: Definition of rotation capacity $R_{0.95}$ employed in the present study

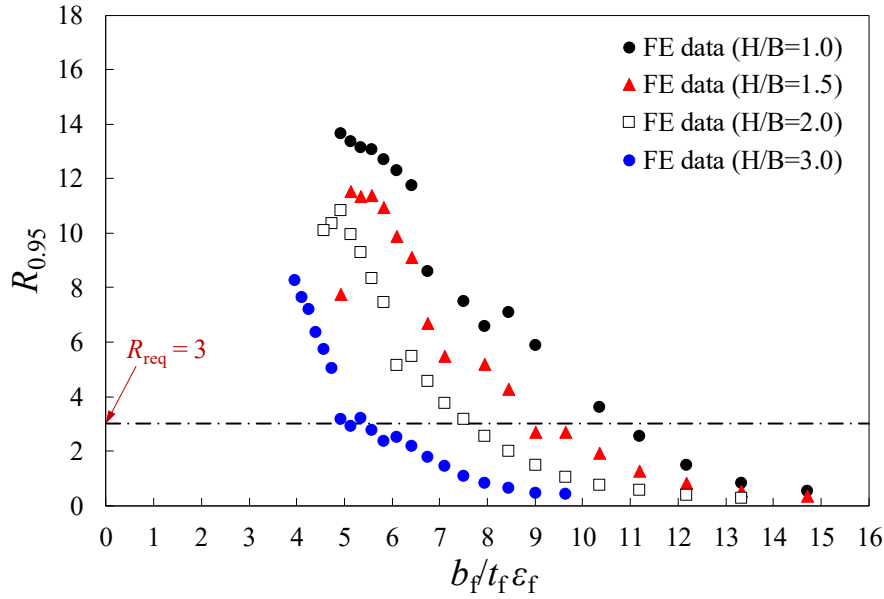
4.4.3 Influence of cross-section aspect ratio

The influence of the cross-section aspect ratio on the normalised ultimate bending moment capacity M_u/M_{pl} and rotation capacity $R_{0.95}$ of S690 steel welded I-sections is discussed in this section. The normalised ultimate bending moments M_u/M_{pl} and rotation capacities $R_{0.95}$ of the S690 steel beams in three-point bending, obtained from the parametric studies, are plotted against the flange slenderness ratio $b_f/t_f \varepsilon_f$ in Figure 4.13(a) and (b), respectively, where b_f is the outstand width, t_f is the flange thickness and $\varepsilon_f = (235/f_{y,f})^{0.5}$, in which $f_{y,f}$ is the yield strength of the flange. The numerically derived data are categorised according to their aspect ratios; the EC3 Class 1 and Class 2 slenderness limits for outstand flanges in compression specified in EN 1993-1-1 (2019) are also indicated in Figure 4.13(a). In addition, the rotation requirement of three ($R_{req} = 3$), which was used in the development of EC3 (Bild et al., 1989; Chan &

Gardner, 2008; European Commission et al., 1998), is also plotted in Figure 4.13(b). It can be seen from Figure 4.13 that lower normalised ultimate bending moment capacities and rotation capacities are achieved for cross-sections with higher aspect ratios for a given flange slenderness; this is caused by the decreasing ability of the web to provide restraint against local buckling to the flange with increasing aspect ratio (Gardner et al., 2019). This observation echoes the numerical results for the other investigated HSS homogeneous and hybrid welded I-sections, and also mirrors the findings for square and rectangular hollow section (i.e. SHS and RHS) members in bending reported by Wang et al. (2016) and Zhou and Long (2016).



(a) Normalised ultimate bending moment (M_u/M_{pl}) versus flange slenderness



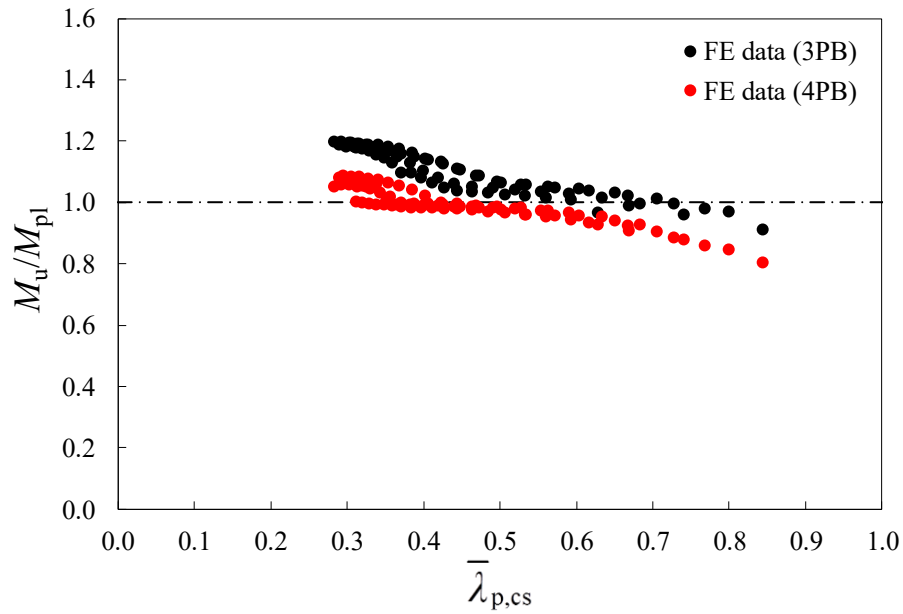
(b) Rotation capacity versus flange slenderness

Figure 4.13: Influence of cross-section aspect ratio on normalised ultimate bending moment M_u/M_{pl} and rotation capacity $R_{0.95}$ of S690 steel welded I-section beams in three-point bending.

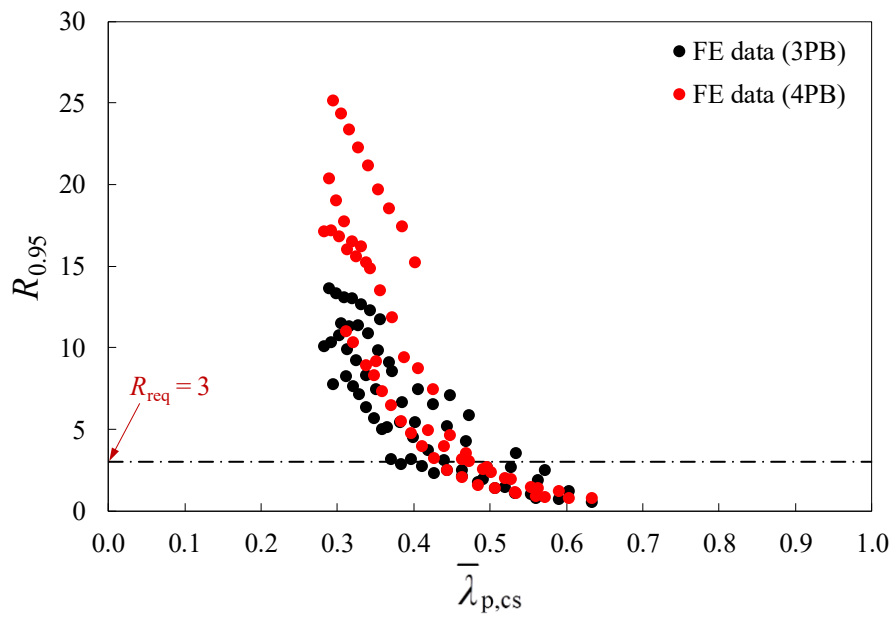
4.4.4 Influence of moment gradient

The influence of moment gradient on the flexural behaviour of welded I-sections is examined by comparing the parametric FE results of the S690 steel beams in three-point (3PB) and four-point (4PB) bending, with the latter providing the most severe case of uniform bending (i.e. no moment gradient) between the loading points. The normalised ultimate bending moments M_u/M_{pl} and rotation capacities $R_{0.95}$ are plotted against the non-dimensional cross-section slenderness $\bar{\lambda}_{p,cs}$ in Figure 4.14(a) and (b), respectively. It can be observed from Figure 4.14(a) that for any given cross-section, the beam in three-point bending achieves a higher normalised ultimate bending moment capacity M_u/M_{pl} than that in four-point bending, due to the beneficial effect of the moment gradient. For a beam in three-point bending, the moment gradient within the beam allows the stiffer cross-sections (due to the lower level of plasticity experienced) of the specimen surrounding the mid-span section where the maximum moment arises to delay the development of local buckling. For a beam in four-point bending, on the other hand, the central part between the two loading points where constant moment is applied and experiences essentially the same level of plasticity along the full length, and hence such restraint to delay local buckling is not available. Similar findings were observed for both the homogeneous and

hybrid welded I-sections, and have been reported by other researchers on SHS, RHS (Wang et al., 2016c) and elliptical hollow section (EHS) beams (Chan & Gardner, 2008; Chen & Young, 2019a, 2019b). As shown in Figure 4.14(b), the beams with stockier cross-sections in four-point bending generally display higher rotation capacities than their counterparts in three-point bending.



(a) Normalised ultimate bending moment



(b) Rotation capacity

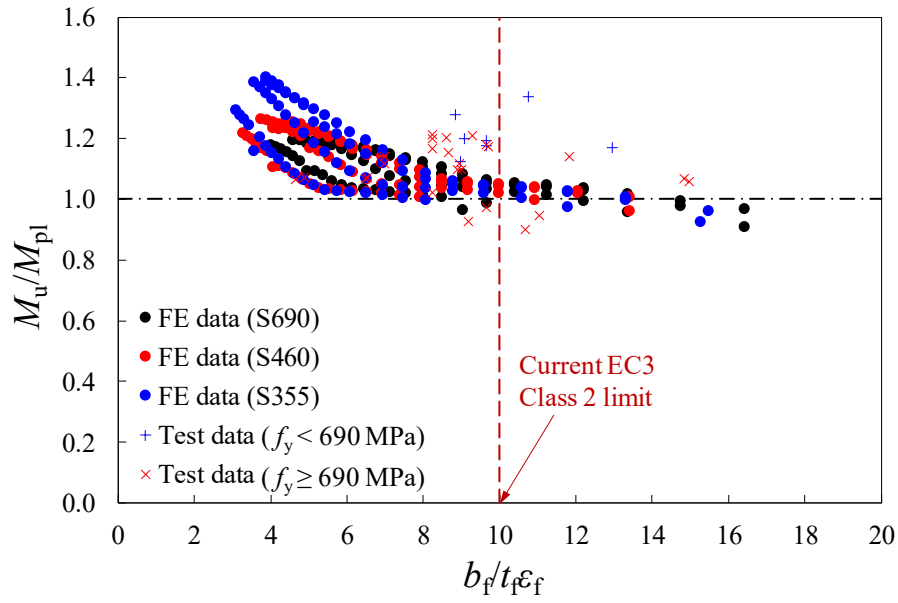
Figure 4.14: Influence of moment gradient on normalised ultimate bending moment M_u/M_{pl} and rotation capacity $R_{0.95}$ of S690 steel welded I-section beams.

4.4.5 Assessment of EC3 slenderness limits for the classification of HSS homogeneous welded I-sections

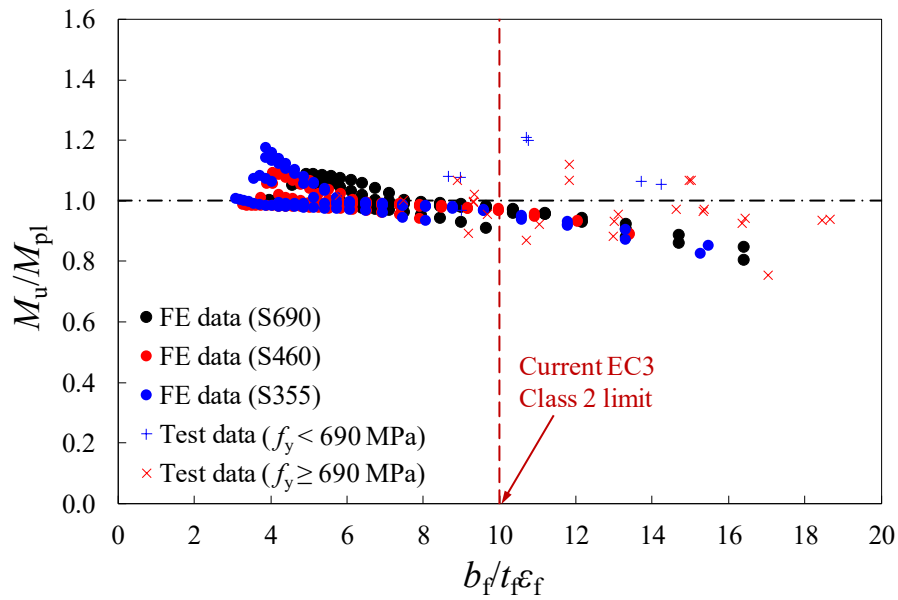
In this subsection, the suitability of the current codified slenderness limits in EN 1993-1-12 (EC3) (2007) for the classification of HSS outstand plate elements in compression and HSS internal plate elements in bending is assessed using the experimental results from the literature and the present study, as summarised in Table 2.1, together with the generated FE results.

4.4.5.1 Class 2 and Class 3 slenderness limits

To assess the applicability of the current EC3 Class 2 slenderness limits for S690 steel outstand elements in compression and internal (i.e. web) elements in bending, the test and numerical ultimate bending moments for the S690 steel welded I-section beams are normalised by the cross-section plastic bending resistances M_{pl} , and plotted against the flange slenderness ($b_f/t_f\epsilon_f$) and web slenderness ($h_w/t_w\epsilon_w$) ratios in Figure 4.15 and Figure 4.16 ((a) for three-point bending and (b) for four-point bending), respectively. The results for the S355 and S460 steel welded I-section beams, together with the current EC3 Class 2 slenderness limits (i.e. $b_f/t_f\epsilon_f = 10$ for outstand elements in compression and $h_w/t_w\epsilon_w = 83$ for internal elements in bending), are also plotted in Figure 4.15 and Figure 4.16 for comparison purposes. It can be seen from Figure 4.15 and Figure 4.16 that the S355, S460 and S690 data points generally follow the same trend, and the current EC3 Class 2 slenderness limits are therefore deemed suitable for the classification of S690 steel welded I-section beams. It should be noted though, that for all steel grades, the data points for the cross-sections that are within the slenderness limits often fall below M_{pl} in the case of four-point bending; as discussed in Section 4.4.4, this is related to the reduced restraint against local buckling afforded to the critical cross-section in the case of uniform bending (compared to three-point bending) and hence the reduced ability to benefit from strain hardening to enable capacities beyond M_{pl} to be achieved.

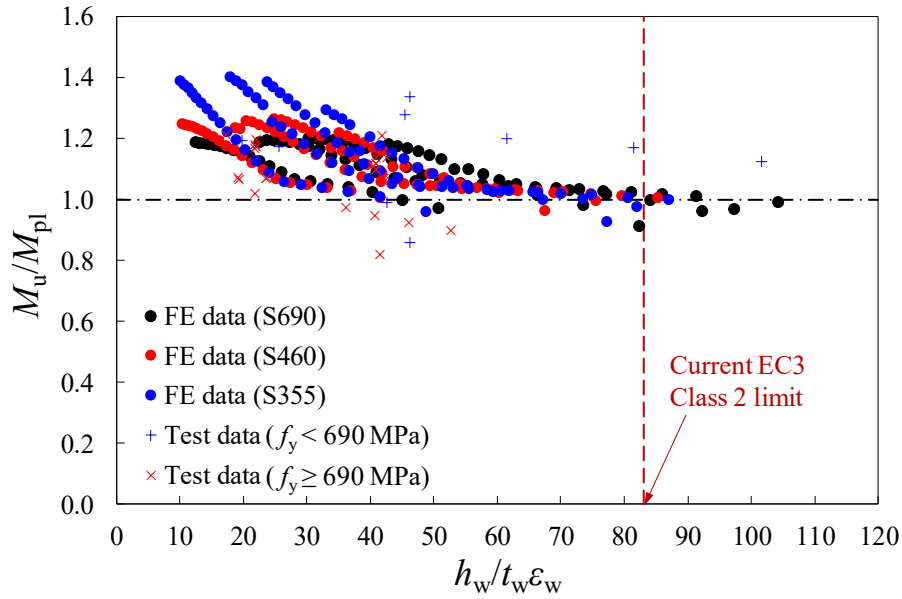


(a) Beam specimens in three-point bending

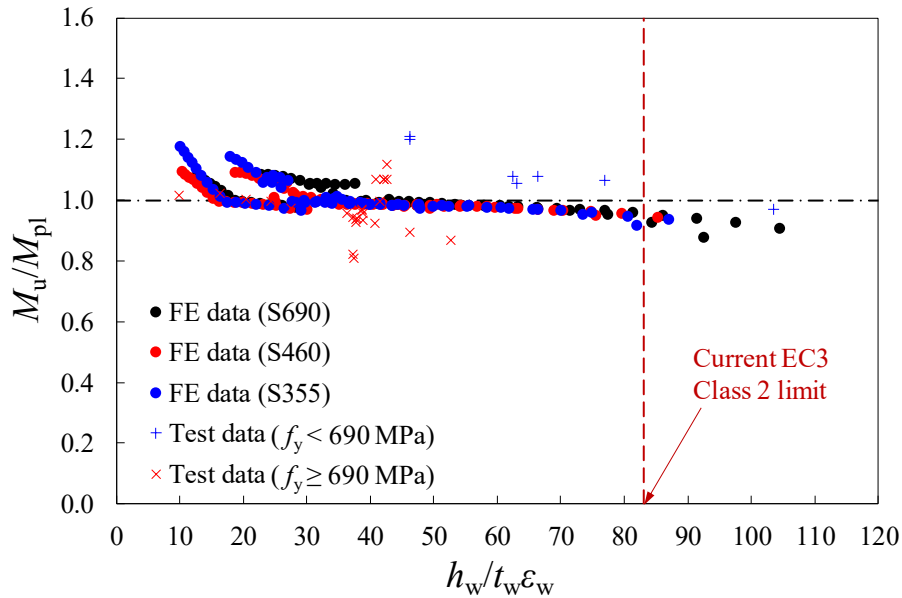


(b) Beam specimens in four-point bending

Figure 4.15: Examination of EC3 Class 2 slenderness limit for outstand elements of homogeneous beams in compression.



(a) Beam specimens in three-point bending

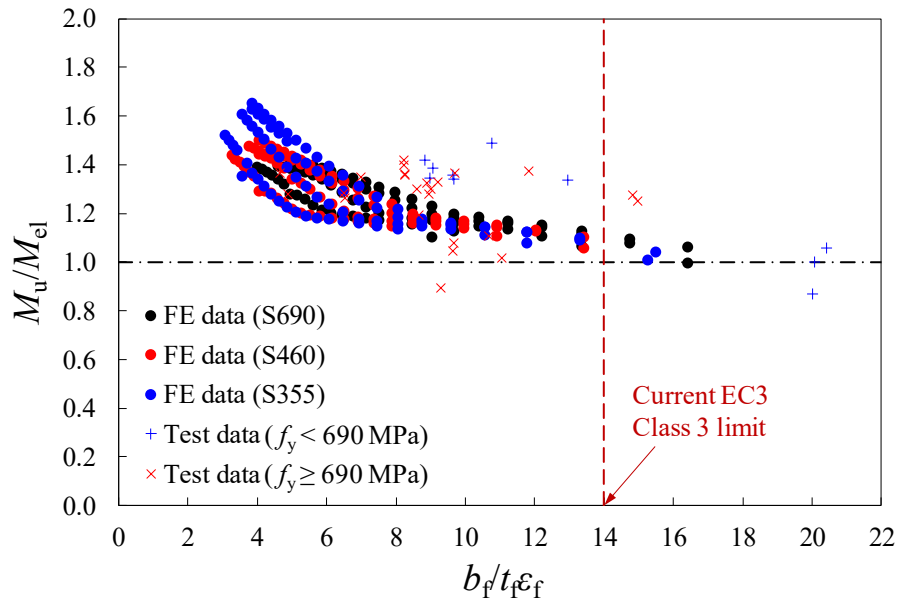


(b) Beam specimens in four-point bending

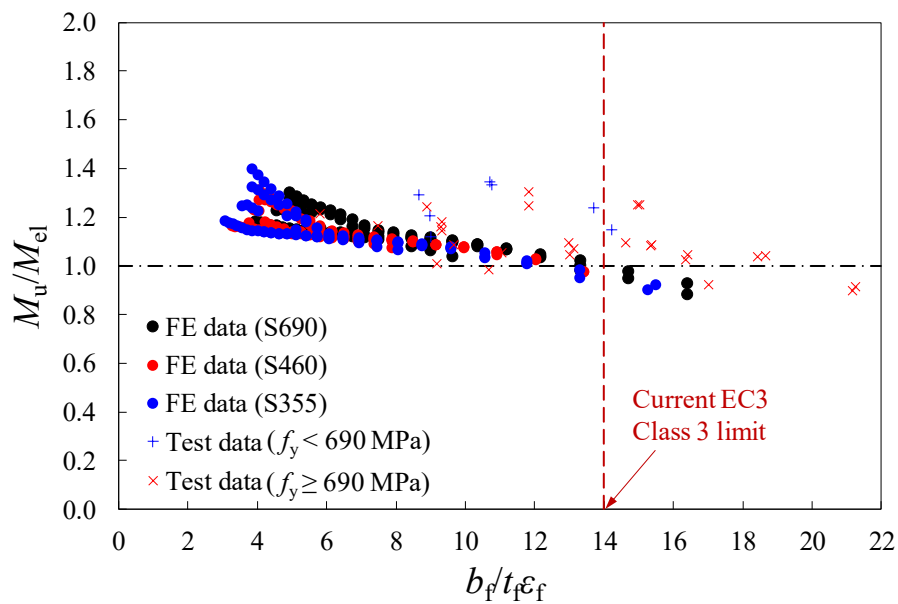
Figure 4.16: Examination of EC3 Class 2 slenderness limit for internal elements of homogeneous beams in bending.

The current EC3 Class 3 slenderness limits for S690 steel outstand elements in compression and internal (i.e. web) elements in bending are also assessed in Figure 4.17 and Figure 4.18, respectively, where the test and numerical ultimate bending moments are now normalised by the cross-section elastic bending resistances M_{el} . The comparisons shown in Figure 4.17 and

Figure 4.18 confirm the suitability of the current EC3 Class 3 slenderness limits for S690 steel welded I-sections subjected to major axis bending.

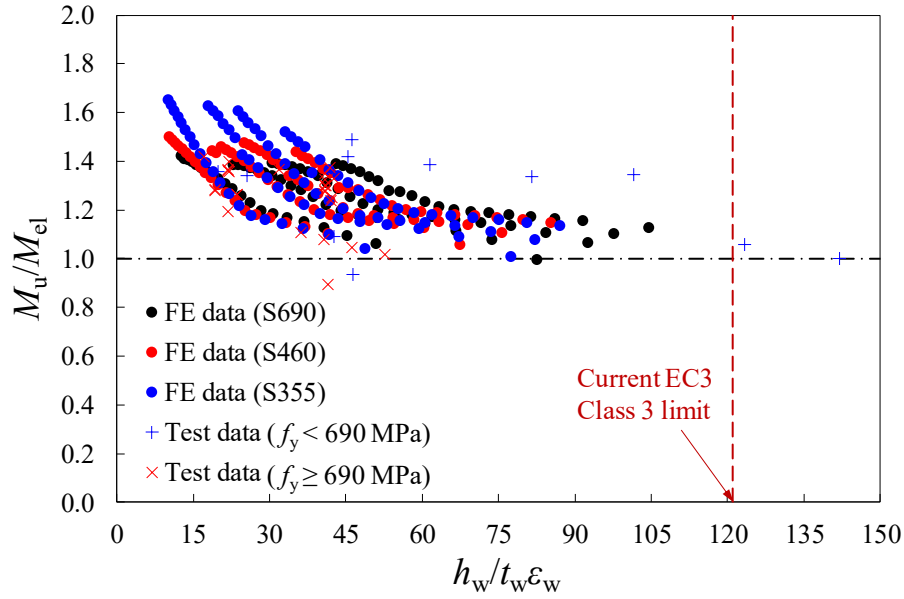


(a) Beam specimens in three-point bending

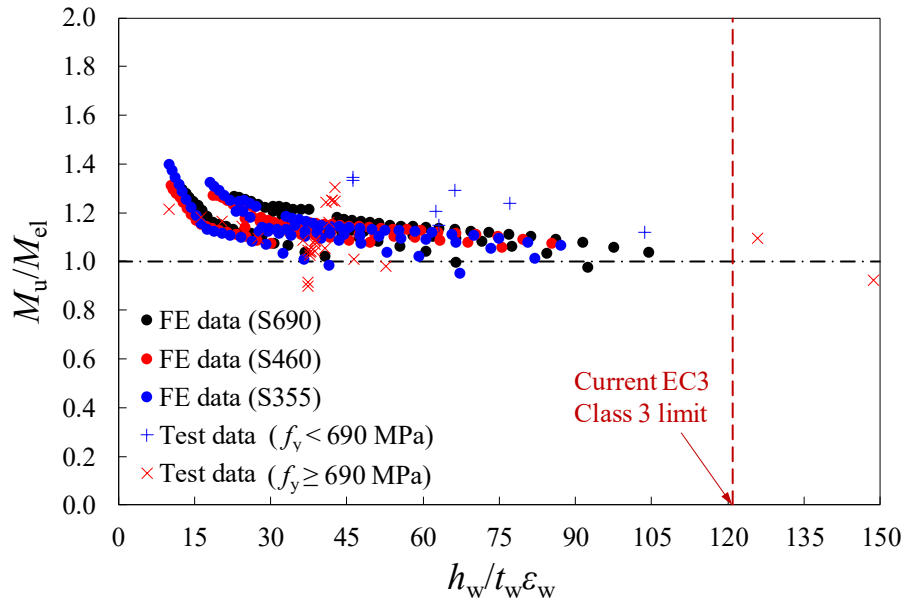


(b) Beam specimens in four-point bending

Figure 4.17: Examination of EC3 Class 3 slenderness limit for outstand elements of homogeneous beams in compression.



(a) Beam specimens in three-point bending



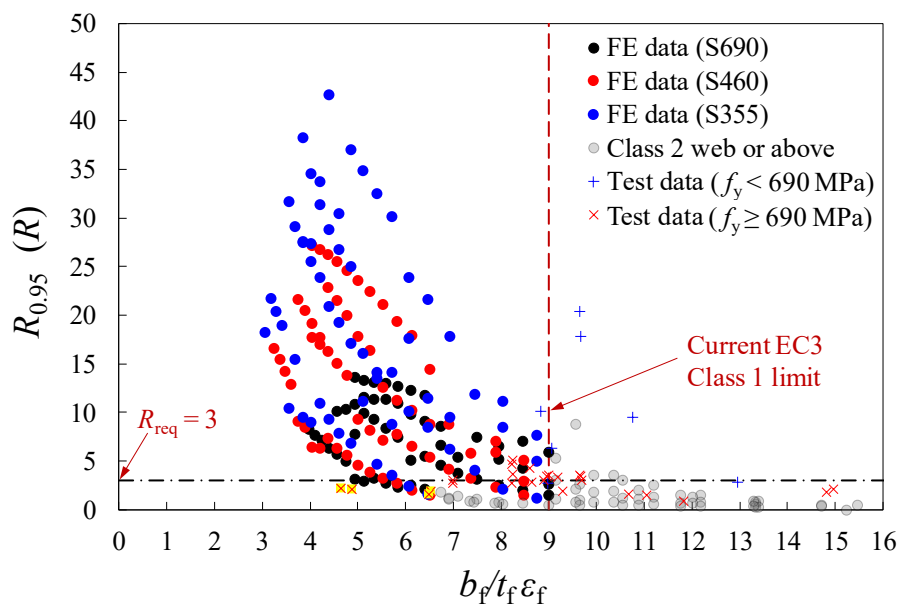
(b) Beam specimens in four-point bending

Figure 4.18: Examination of EC3 Class 3 slenderness limit for internal elements of homogeneous beams in bending.

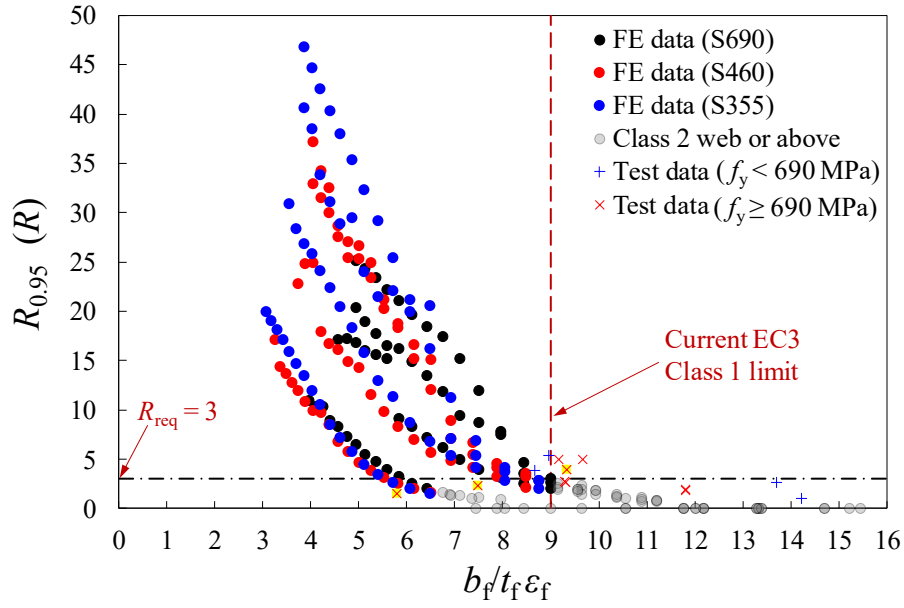
4.4.5.2 Class 1 slenderness limits

To assess the current EC3 Class 1 slenderness limits, the rotation capacities from both the experiments (R) and FE models ($R_{0.95}$) are plotted against the flange slenderness ($b_f/t_f \epsilon_f$) and web slenderness ($h_w/t_w \epsilon_w$) ratios in Figure 4.19 and Figure 4.20 respectively, together with the current EC3 Class 1 slenderness limits (i.e. $b_f/t_f \epsilon_f = 9$ for outstand elements in compression and

$h_w/t_w\epsilon_w = 72$ for internal elements in bending) and the rotation capacity requirement of $R_{\text{req}} = 3$ for Class 1 cross-sections. Note that the use of R (instead of $R_{0.95}$) was retained for the experiments since rotation capacities were typically reported in the literature using this measure. It can be seen from Figure 4.19 and Figure 4.20 that the current EC3 Class 1 slenderness limits appear somewhat unconservative for both NSS and HSS welded I-sections. Note that when the outstand flange slenderness limits are being assessed (i.e. in Figure 4.19), the data points corresponding to the cross-sections where the web is Class 2 or above are shaded in grey, since both the flange and web must be Class 1 for plastic design to be applied. The same approach is used when assessing the web slenderness limits in Figure 4.20. Lower slenderness limits of $b_f/t_f\epsilon_f = 8$ for outstand elements in compression and $h_w/t_w\epsilon_w = 60$ for internal plate elements in bending are thus proposed to provide improved predictions of when $R_{\text{req}} = 3$ can be achieved for both NSS and HSS welded I-sections. The slightly stricter proposed slenderness limits are considered to be essential for HSS due to the lack of historical precedent in applying plastic design to HSS structures. The rotation capacities of the newly classified Class 1 cross-sections are plotted against their flange and web slenderness ratios in Figure 4.21 and Figure 4.22, together with the proposed Class 1 slenderness limits for outstand flanges (in compression) and internal elements (in bending), which provide predictions of when $R_{\text{req}} = 3$ can be achieved. Note that the data points shaded in yellow indicate tests carried out in the present study that were terminated prior to reaching the full rotation capacity R .

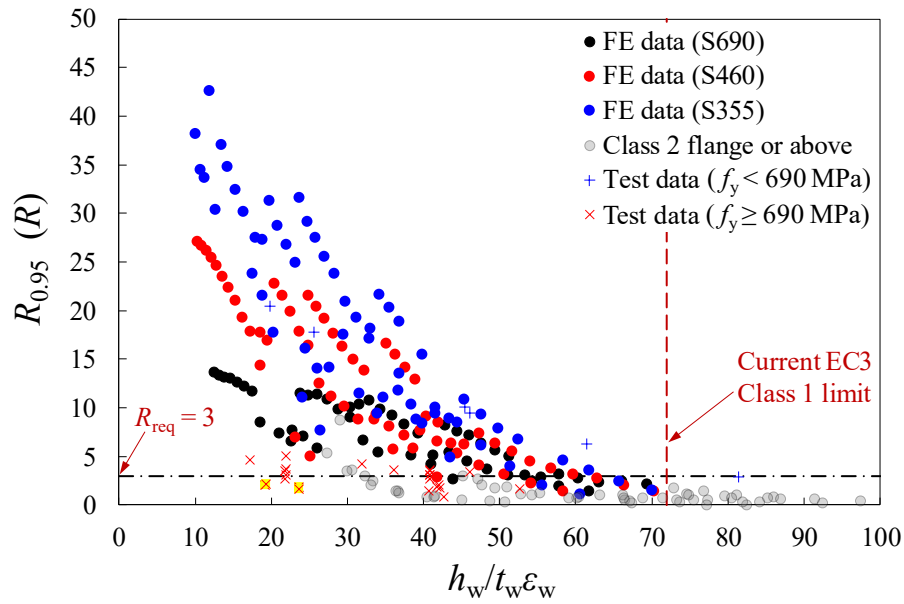


(a) Beam specimens in three-point bending

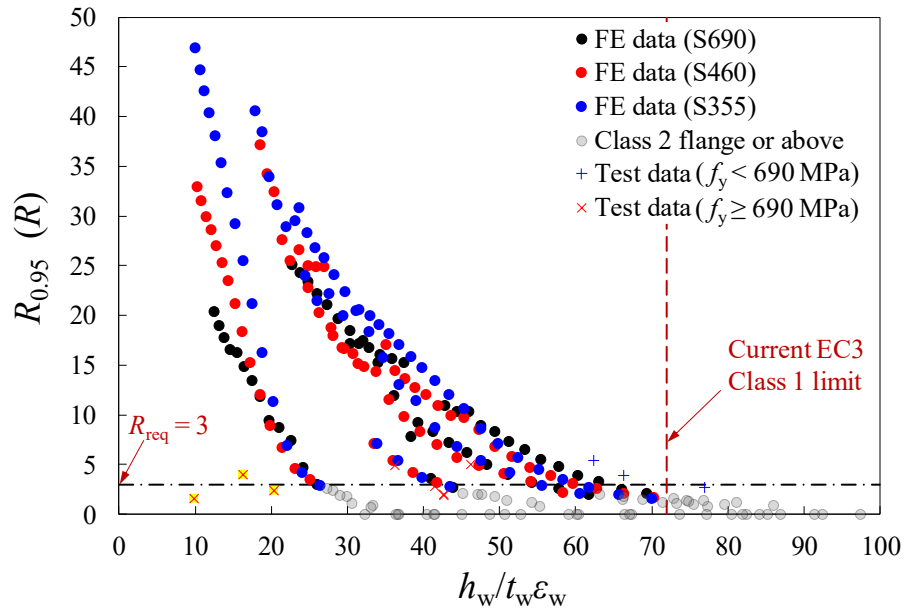


(b) Beam specimens in four-point bending

Figure 4.19: Examination of EC3 Class 1 slenderness limit for outstand elements in compression in homogeneous welded I-section beams.

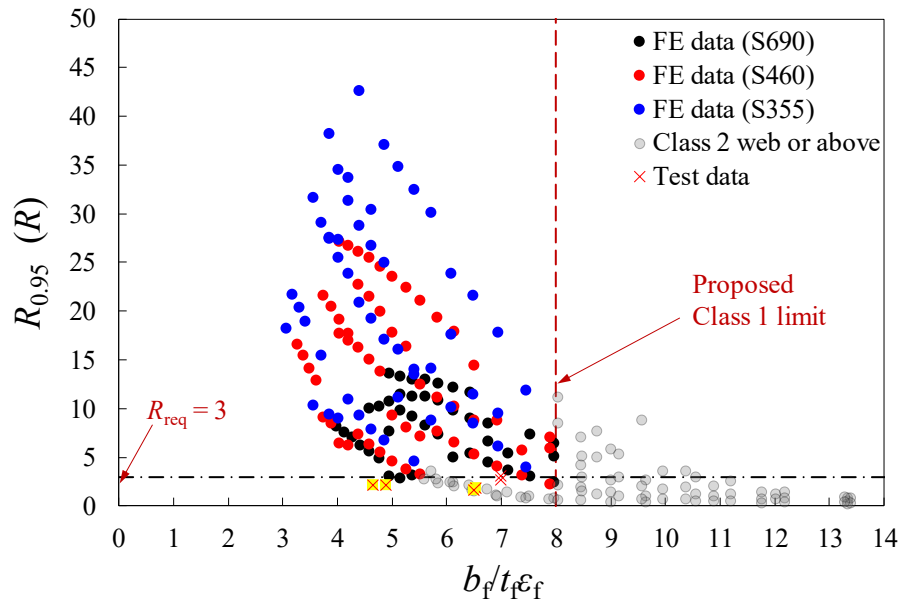


(a) Beam specimens in three-point bending

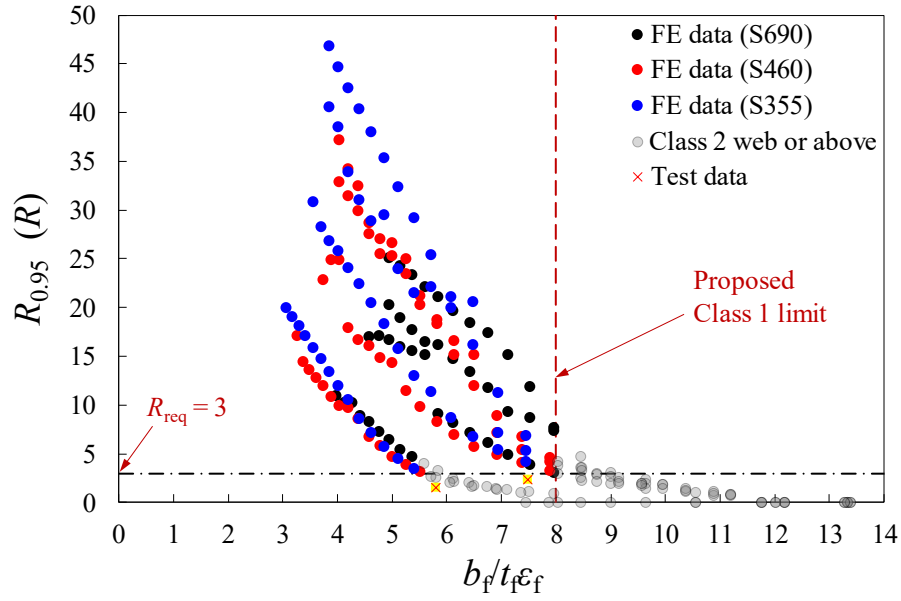


(b) Beam specimens in four-point bending

Figure 4.20: Examination of EC3 Class 1 slenderness limit for internal elements in bending in homogeneous welded I-section beams.

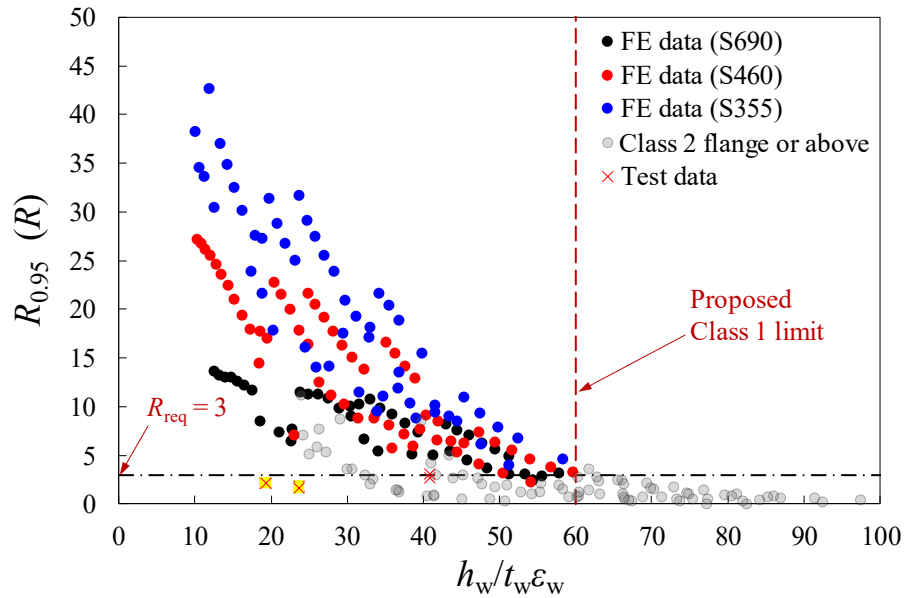


(a) Beam specimens in three-point bending

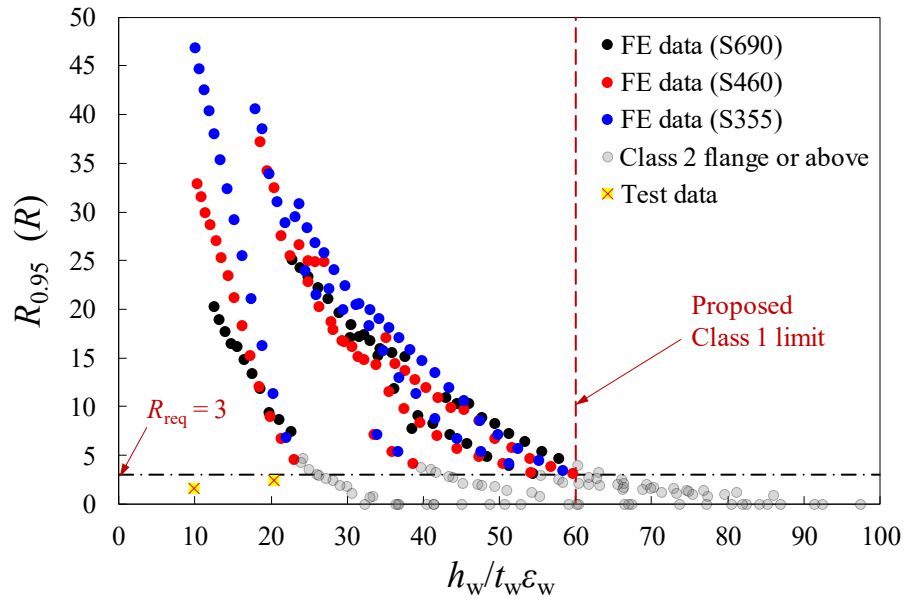


(b) Beam specimens in four-point bending

Figure 4.21: Examination of the proposed Class 1 slenderness limit for outstand elements in compression in homogeneous welded I-section beams.



(a) Beam specimens in three-point bending



(b) Beam specimens in four-point bending

Figure 4.22: Examination of the proposed Class 1 slenderness limit for internal elements in bending in homogeneous welded I-section beams.

4.4.6 Analysis of results and design recommendations for HSS hybrid welded I-section beams

In this section, the structural performance of HSS hybrid welded I-section beams subjected to major axis bending is discussed and the applicability of the EC3 design rules to these sections is evaluated based on the established test and FE results.

4.4.6.1 Bending resistance

The elastic bending moment M_{el} of the investigated HSS hybrid welded I-sections in major axis bending is defined herein as the bending moment at which the stress at the extreme outer-fibre of the flanges reaches the material yield strength $f_{y,f}$ (i.e. the strain at the extreme outer-fibre of the flanges reaches the yield strain of the flange plates $\epsilon_{y,f}$). The strain and stress distributions throughout the depth of an HSS hybrid welded I-section at the defined elastic bending moment M_{el} are shown in Figure 4.23, assuming a linear strain distribution and elastic-perfectly-plastic material behaviour for both the flanges and the web. From the presented stress distribution, M_{el} can be determined from:

$$M_{el} = W_{el,f} f_{y,f} + W_{el,w} f_{y,w} + W_{pl,w} f_{y,w} \quad (4.5)$$

where $W_{el,f}$ and $W_{el,w}$ are the elastic section moduli associated with the flanges and the elastic portion of the web, respectively, as given by:

$$W_{el,f} = \frac{Bt_f^3/3 + Bt_f(H - t_f)^2}{H} \quad (4.6)$$

and

$$W_{el,w} = \frac{1}{6} t_w H_{el,w}^2, \quad (4.7)$$

$W_{pl,w}$ is the plastic section modulus associated with the plastic portion of the web, as given by:

$$W_{pl,w} = 2t_w \cdot \left(\frac{H}{2} - \frac{H_{el,w}}{2} - t_f \right) \cdot \left(t_f - \frac{H}{2} + H_{el,w} \right), \quad (4.8)$$

$f_{y,f}$ and $f_{y,w}$ are the yield strengths of the flange and the web plates, respectively, and $H_{el,w}$ is the height of the elastic portion of the web, as illustrated in Figure 4.23 and given by:

$$H_{el,w} = \frac{f_{y,w}/E_w}{f_{y,f}/E_f} H \quad (4.9)$$

where E_f is the Young's modulus of the flange plate and E_w is the Young's modulus of the web plate.

Similar to homogeneous welded I-sections, the plastic bending moment M_{pl} for the investigated HSS hybrid welded I-sections in major axis bending is defined as the moment at which the cross-section is fully yielded, i.e. the flange plates and the web plate are at their corresponding yield strengths.

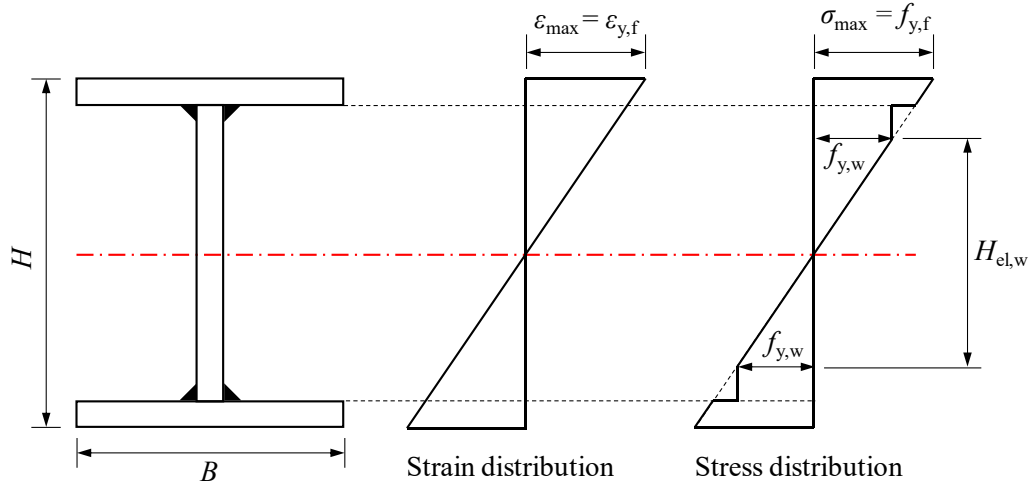


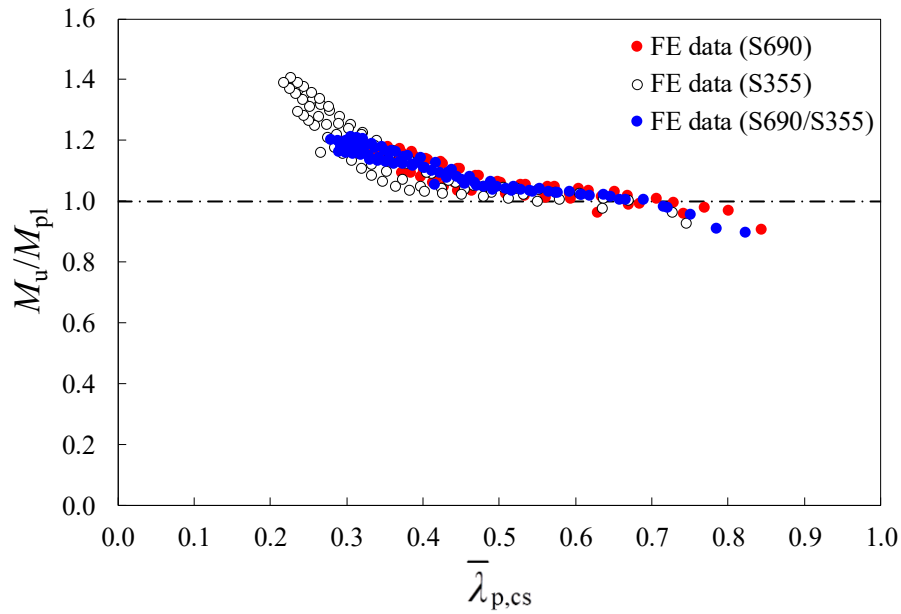
Figure 4.23: Strain and stress distributions for an HSS hybrid welded I-section in major axis bending at the defined elastic bending moment M_{el} .

The normalised ultimate bending resistances M_u/M_{pl} are plotted against the non-dimensional cross-section slenderness $\bar{\lambda}_{p,cs}$ in Figure 4.24(a) for the hybrid beams composed of S690 steel flanges and an S355 steel web, as well as the S355 steel and S690 steel homogeneous beams, in three-point bending. The equivalent graph for the hybrid beams composed of S690 steel flanges and an S460 steel web, as well as the S460 steel and S690 steel homogeneous beams in three-point bending is shown in Figure 4.24(b). Note that the non-dimensional cross-section slenderness $\bar{\lambda}_{p,cs}$ for hybrid beams is defined by Equation (4.10):

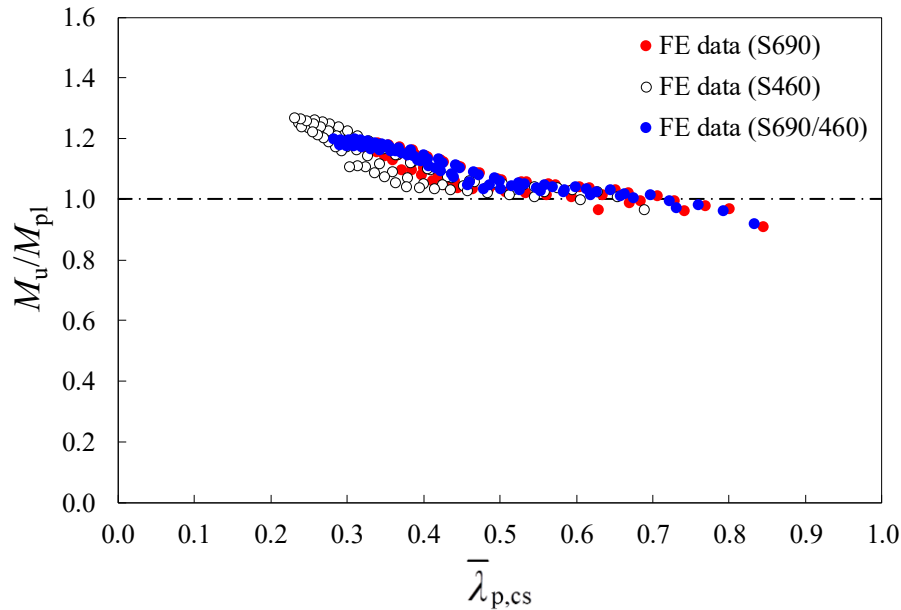
$$\bar{\lambda}_{p,cs} = \sqrt{\frac{M_{el}}{M_{cr}}} = \sqrt{\frac{M_{el}}{\sigma_{cr} W_{el}}} \quad (4.10)$$

where M_{el} is the elastic bending moment calculated from Equations (4.5)-(4.9), σ_{cr} is the elastic local buckling stress of the full cross-section determined using the finite strip software CUFSM (Schafer & Adany, 2006) and W_{el} is cross-sectional elastic section modulus. It can be observed from Figure 4.24 that the data points for the hybrid beams generally follow a similar trend to that of the homogeneous beams, especially for the beams with more slender cross-sections, for which failure occurred prior to the maximum strain within the critical region of the beam specimens reaching the strain hardening strain ε_{sh} . For the beams with stockier cross-sections, where capacity benefits are derived from strain hardening, the normalised ultimate bending

resistances M_u/M_{pl} for the hybrid beams are more similar to those of the homogeneous beams made of the higher steel grade (i.e. the steel grade of the flange plates of the hybrid beams), since the strain hardening contribution is largely dependent on the material characteristics of the flange plates.



(a) Hybrid beams (composed of S690 steel flanges and an S355 steel web) and S355 steel and S690 steel homogeneous beams

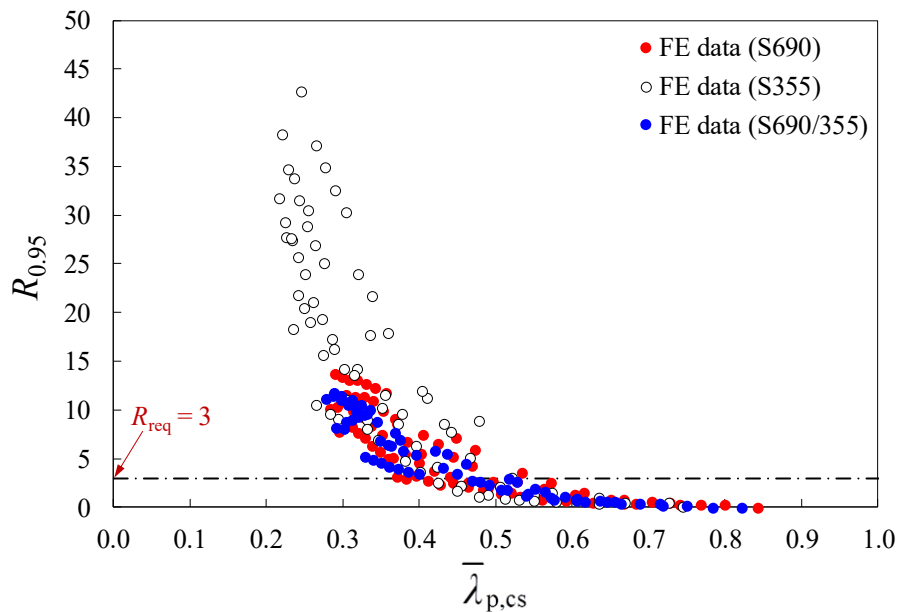


(b) Hybrid beams (composed of S690 steel flanges and an S460 steel web) and S460 steel and S690 steel homogeneous beams

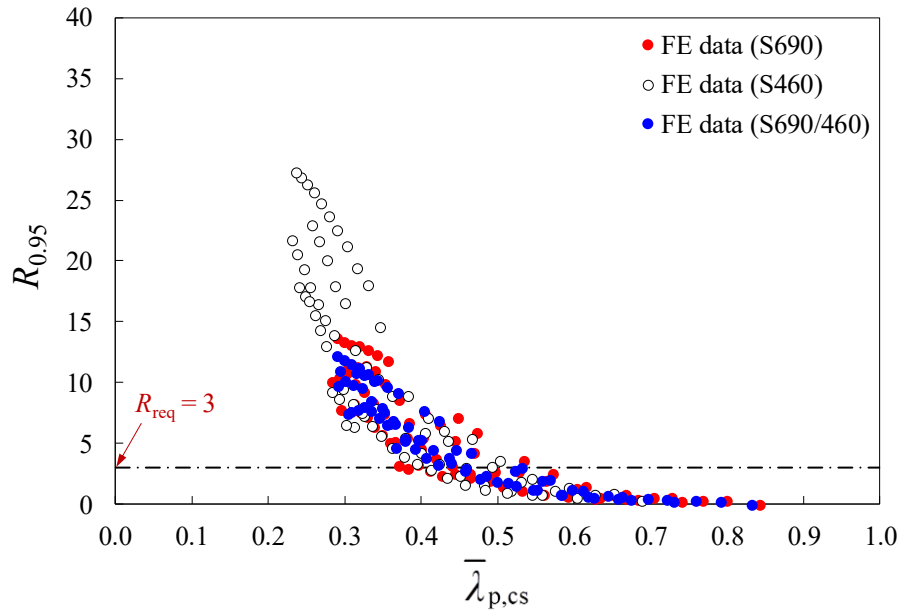
Figure 4.24: Comparison of normalised ultimate bending resistances M_u/M_{pl} for homogeneous and hybrid welded I-section beams under three-point bending.

4.4.6.2 Rotation capacity

The rotation capacities of the investigated homogeneous and hybrid beams under three-point bending are plotted against the non-dimensional slenderness $\bar{\lambda}_{p,cs}$ in Figure 4.25. Similar to the behaviour observed for the normalised bending resistances, the rotation capacities of the HSS hybrid beams generally follow the same trend as that of the homogeneous beams, especially for the beams with the more slender cross-sections. The hybrid beams with the stockier cross-sections (i.e. with $\bar{\lambda}_{p,cs}$ lower than about 0.4) exhibit lower rotation capacities compared with their homogeneous counterparts made of the lower steel grade (i.e. S355 or S460 steel). Compared to the HSS homogeneous beams, the HSS hybrid beams subjected to the major axis bending experience gradual plastification of the web (i.e. initiating from the outer-fibre of the web plate) prior to the yielding of the flanges, which reduces the ability of the web to provide rotational restraint to the flanges against local buckling. The rotation capacities of the HSS hybrid beams are thus also somewhat lower than those of the counterpart homogeneous beams made of the higher steel grade; this is more pronounced for the hybrid beams that have a higher yield strength ratio of $f_{y,f}/f_{y,w}$, as seen in Figure 4.25(a). It is recommended that for HSS hybrid welded I-sections, the yield strength of the flange plates should be no higher than twice the yield strength of the web (i.e. $f_{y,f} \leq 2f_{y,w}$) (EN 1993-1-5, 2006), since hybrid girders with greater ratios of $f_{y,f}/f_{y,w}$ show significantly reduced rotation capacities.



(a) Hybrid beams (composed of S690 steel flanges and an S355 steel web) and S355 steel and S690 steel homogeneous beams

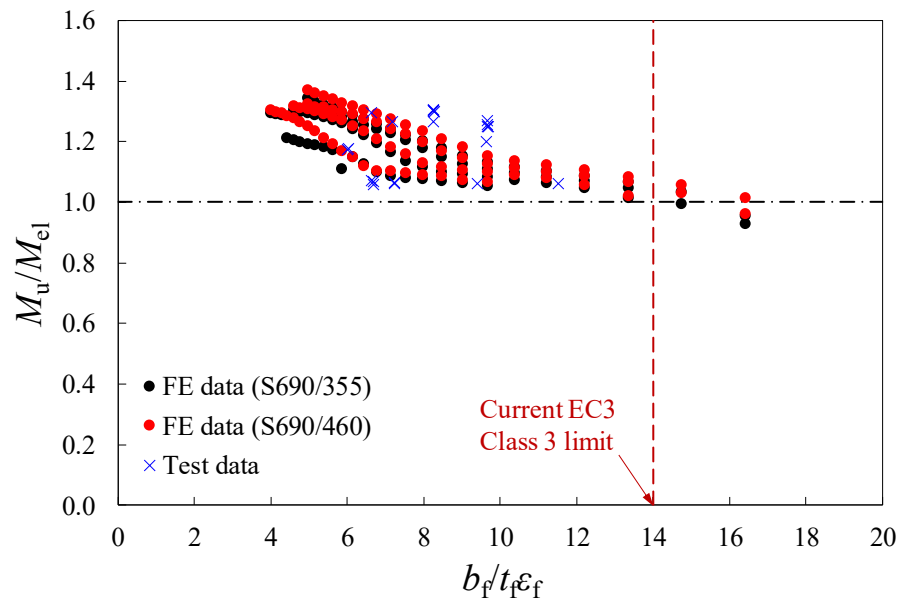


(b) Hybrid beams (composed of S690 steel flanges and an S460 steel web) and S460 steel and S690 steel homogeneous beams

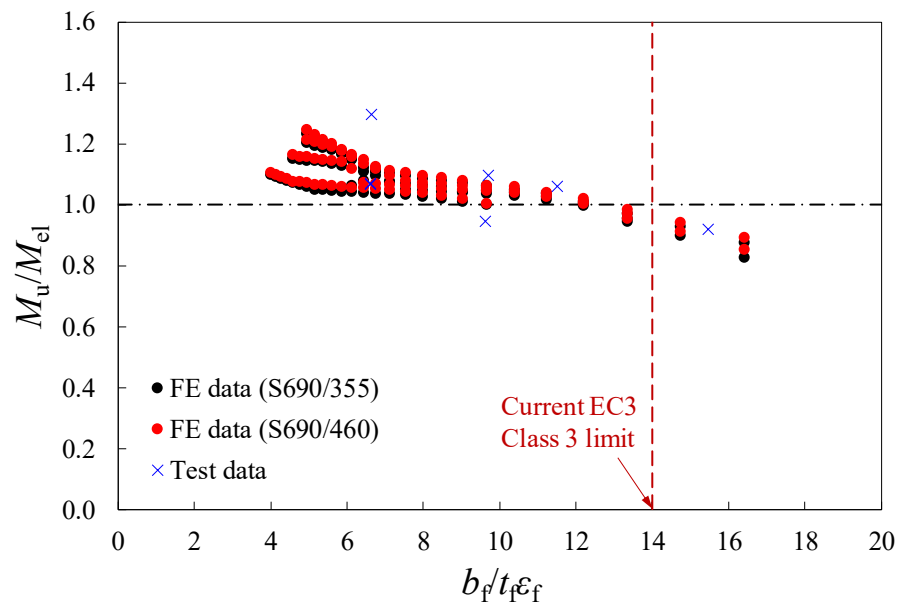
Figure 4.25: Comparison of rotation capacities $R_{0.95}$ for homogeneous and hybrid welded I-section beams in three-point bending.

4.4.6.3 Assessment of EC3 slenderness limits for the classification of HSS hybrid welded I-sections

The test and numerical ultimate bending moments of the hybrid welded I-section beams M_u are normalised by the cross-section elastic bending moment capacities M_{el} (calculated according to Equations (4.5)-(4.9)), and plotted against the flange slenderness (b_f/t_{ef}) and web slenderness (h_w/t_{we}) ratios in Figure 4.26 and Figure 4.27 respectively. The Class 3 slenderness limits for outstand flanges in compression and internal webs in bending set out in EN 1993-1-12 (2007) are also plotted in Figure 4.26 and Figure 4.27 respectively. The comparisons show that the current EC3 Class 3 slenderness limits for HSS welded I-sections are also applicable to HSS hybrid welded I-sections.

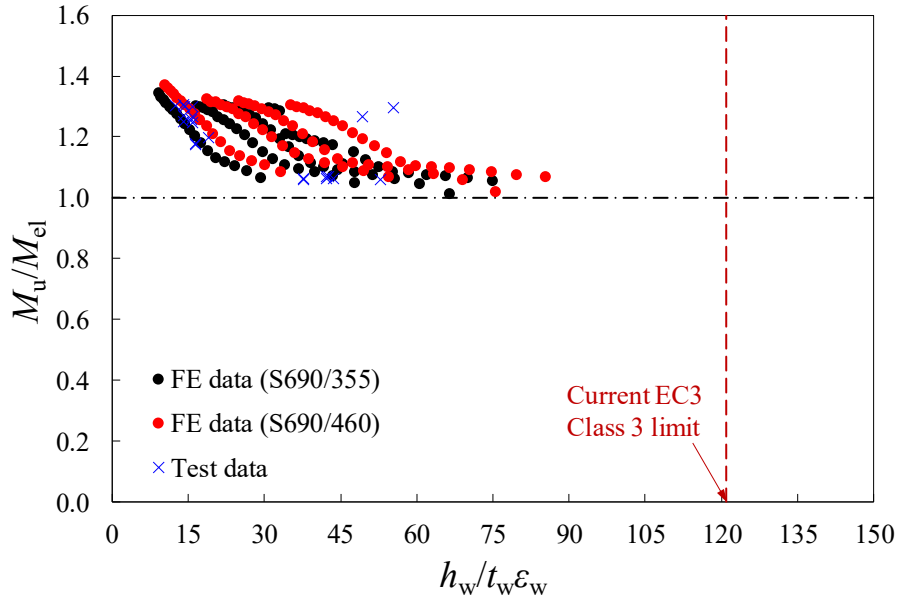


(a) Beam specimens in three-point bending

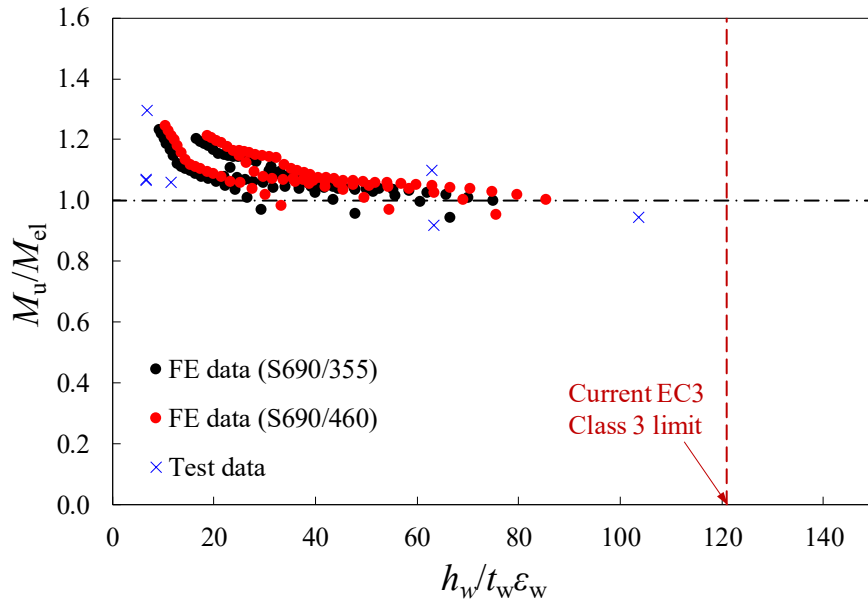


(b) Beam specimens in four-point bending

Figure 4.26: Examination of EC3 Class 3 slenderness limit for outstand flanges in compression in HSS hybrid beams.



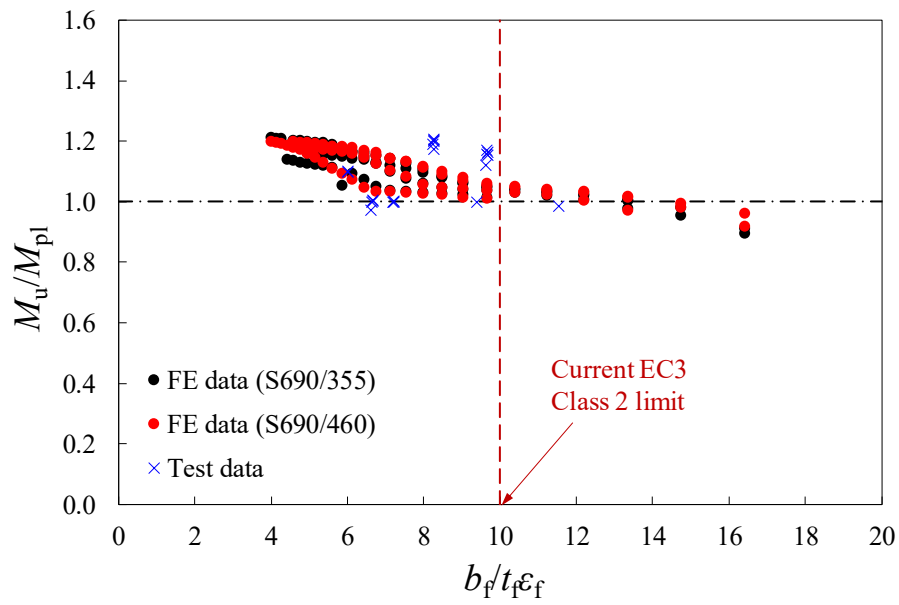
(a) Beam specimens in three-point bending



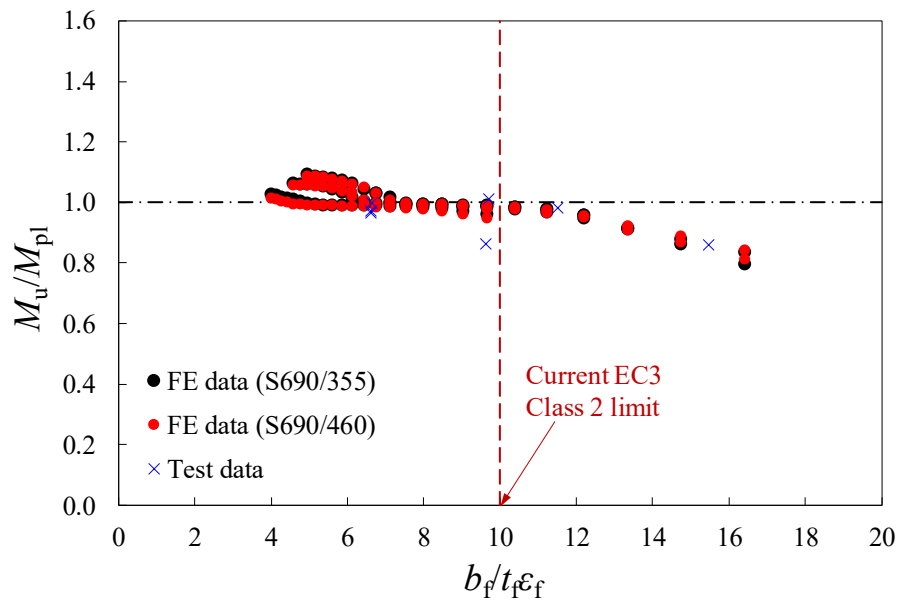
(b) Beam specimens in four-point bending

Figure 4.27: Examination of EC3 Class 3 slenderness limit for internal plate elements in bending in HSS hybrid beams.

Similarly, the current EC3 Class 2 slenderness limits set out in EN 1993-1-12 (2007) are examined in Figure 4.28 and Figure 4.29, where the test and numerical ultimate bending moments M_u for the HSS hybrid beams in major axis bending are now normalised by the cross-section plastic bending moments M_{pl} . Again, the comparisons confirm the suitability of the current EC3 Class 2 slenderness limits for HSS hybrid welded I-sections in major axis bending.

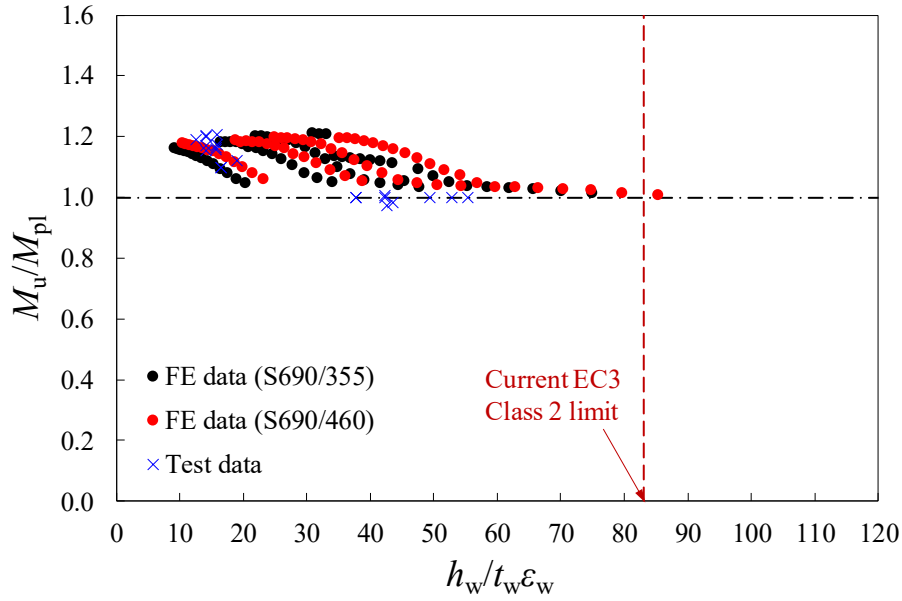


(a) Beam specimens in three-point bending

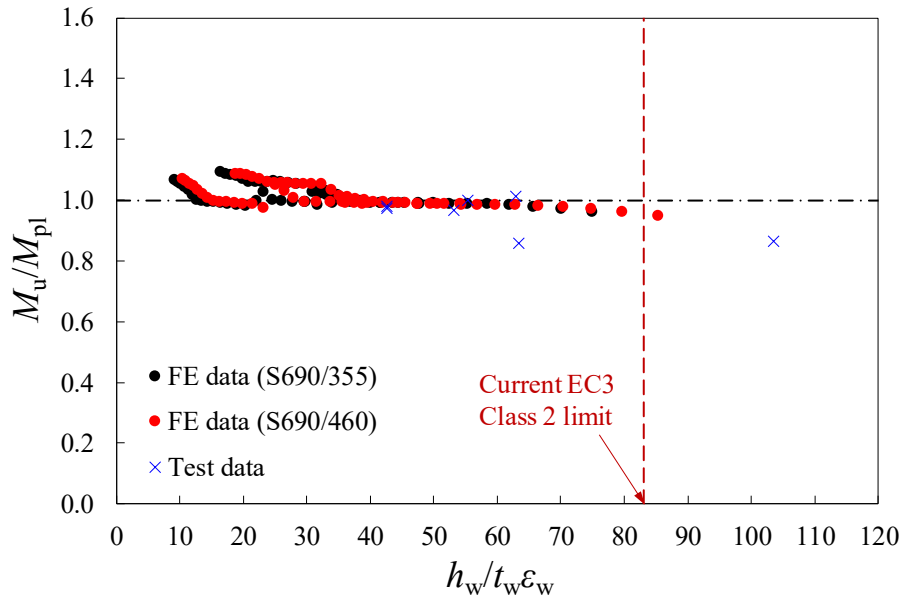


(b) Beam specimens in four-point bending

Figure 4.28: Examination of EC3 Class 2 slenderness limit for outstand flanges in compression in HSS hybrid beams.



(a) Beam specimens in three-point bending

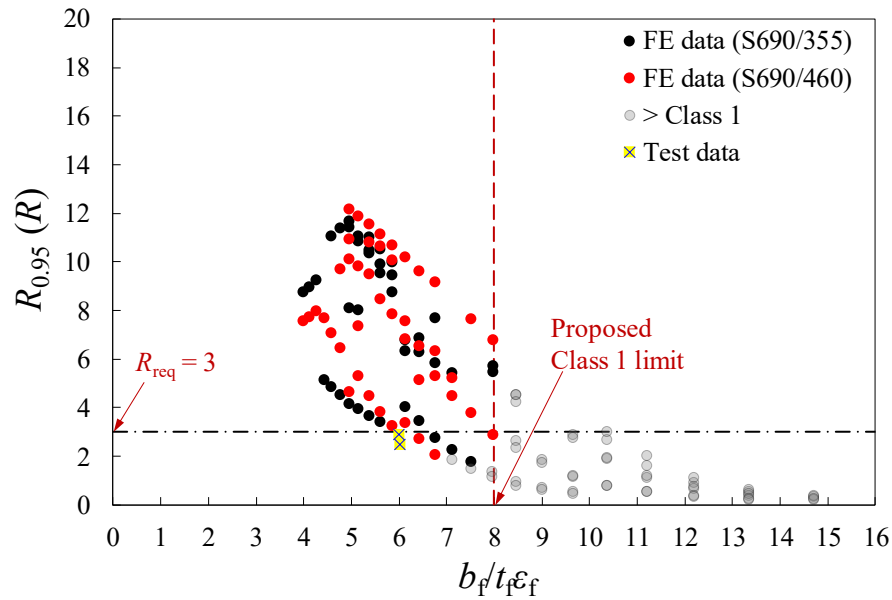


(b) Beam specimens in four-point bending

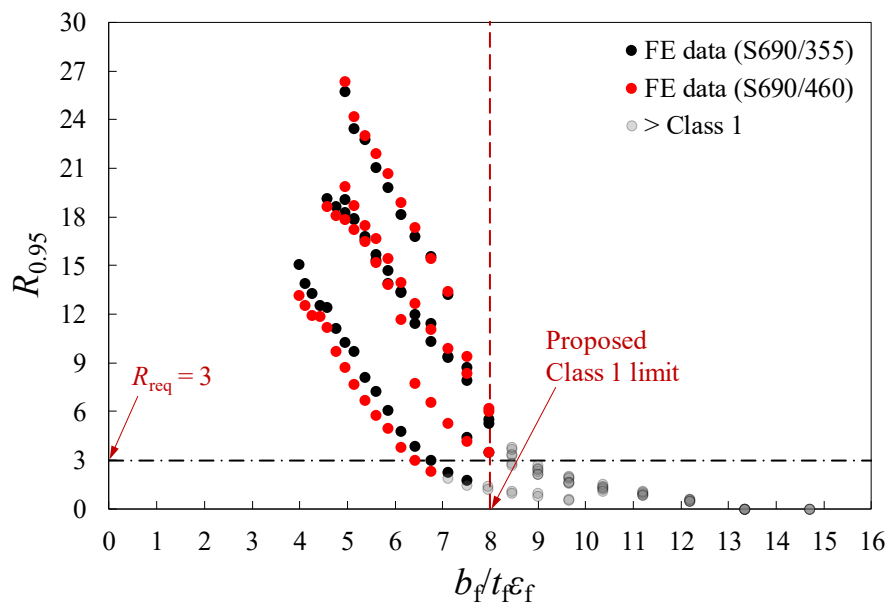
Figure 4.29: Examination of EC3 Class 2 slenderness limit for internal plate elements in compression in HSS hybrid beams.

Finally, the rotation capacities from the experiments (R) and FE models ($R_{0.95}$) of the HSS hybrid beams are plotted against the flange slenderness ratio $b_f/t_f\epsilon_f$ in Figure 4.30 and the web slenderness ratio $h_w/t_w\epsilon_w$ in Figure 4.31. The proposed more strict Class 1 slenderness limits for HSS homogeneous welded I-section beams are also plotted in Figure 4.30 and Figure 4.31.

The comparisons reveal that the proposed Class 1 slenderness limits are also suitable for the classification of HSS hybrid welded I-section beams in major axis bending.

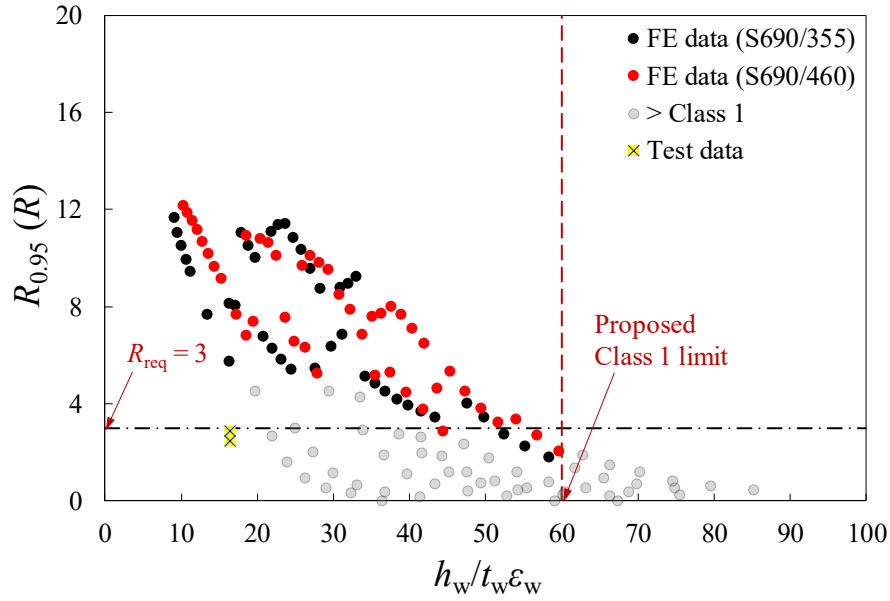


(a) Beam specimens in three-point bending

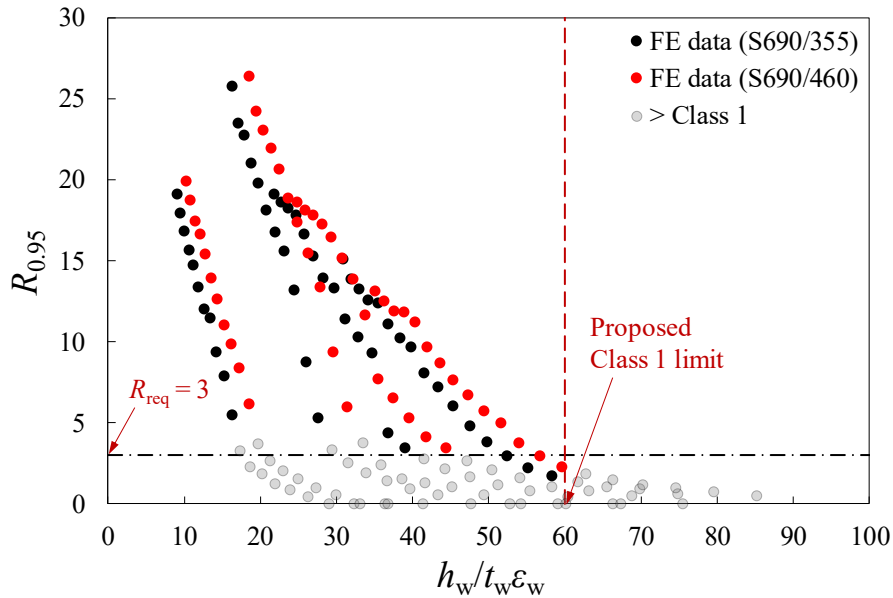


(b) Beam specimens in four-point bending

Figure 4.30: Examination of the proposed Class 1 slenderness limit for outstand flanges in compression in HSS hybrid beams.



(a) Beam specimens in three-point bending



(b) Beam specimens in four-point bending

Figure 4.31: Examination of the proposed Class 1 slenderness limit for internal plate elements in bending in HSS hybrid beams.

4.5 Conclusions

Testing, finite element modelling and design aspects of high strength steel (HSS) beams have been described in this chapter. An experimental programme, which included a total of six in-plane major axis three-point bending tests on both HSS homogeneous and hybrid welded I-

sections, was carried out. The test setup, including the use of a bespoke restraint system to prevent lateral torsional buckling and the employment of DIC to monitor the strain histories of the most heavily loaded regions of the beam specimens, as well as the test results have been carefully presented and analysed. The test results obtained from the present study, as well as those collected from the literature, were subsequently employed in a parallel numerical modelling programme for the validation of FE models. An extensive parametric study was then carried out to generate additional structural performance data for the assessment of the influence of the material grade, cross-section aspect ratio and moment gradient on the flexural behaviour of HSS welded I-sections. The obtained test and FE results were also used to evaluate the applicability of the current EC3 cross-section classification limits (EN 1993-1-1, 2019; EN 1993-1-12, 2007) to both HSS homogeneous and hybrid welded I-sections subjected to major axis bending. It was confirmed that the current EC3 Class 2 and 3 slenderness limits are suitable to classify the outstand flanges (in compression) and internal webs (in bending) of both HSS homogeneous and hybrid welded I-sections. To satisfy the rotation capacity requirement of 3, stricter Class 1 slenderness limits for outstand flanges in compression (i.e. $b_f/t_f \varepsilon_f = 8$) and internal webs in bending ($h_w/t_w \varepsilon_w = 60$) were proposed; the use of the new limits is considered to be essential to allow the plastic design of HSS homogeneous and hybrid systems.

Chapter 5

Welded I-section beam-columns

5.1 Introduction

This chapter presents a comprehensive numerical investigation into the structural behaviour and design of welded I-section beam-columns made of steel grades ranging from S235 to S960. Finite element (FE) models that can replicate the structural response of NSS and HSS welded I-section beam-columns were developed and validated against experimental results collected from the literature (Ma et al., 2017b; Yang et al., 2018). Upon validation, the FE models were employed in extensive parametric studies to generate an FE data pool covering a broad range of cross-section geometries, steel grades, member slendernesses and loading combinations. The generated FE results, as well as the existing test results, were used to evaluate the accuracy of the codified design rules for NSS and HSS welded I-section beam-columns set out in the European Standards EN 1993-1-1 (2005, 2019) and EN 1993-1-12 (2007), and the American Specification AISC 360-16 (2016). Shortcomings in the existing design rules were revealed, which promoted the establishment of a new design approach for welded I-section beam-columns with varying steel grades up to S960. The design proposals are based on newly developed flexural buckling curves for determining the column buckling strengths (see Chapter 3) and the Continuous Strength Method (CSM) (Yun et al., 2018a; Yun & Gardner, 2018) for calculating the cross-sectional bending resistances, serving as the end points of the design interaction curves for beam-columns. The proposed method is shown to provide significantly improved accuracy and consistency in the resistance predictions of welded I-section beam-columns compared with the existing codified design provisions. Finally, the reliability of the

proposed design approach is assessed by performing statistical analyses according to EN 1990 (2002).

5.2 Numerical modelling programme

A comprehensive numerical modelling programme was performed to generate extensive numerical data on welded I-section beam-columns made of NSS and HSS using the FE analysis package ABAQUS (2018). The basic modelling assumptions and validation of the FE models are described in Sections 5.2.1 and 5.2.2, respectively, while the parametric studies are reported in Section 5.2.3.

5.2.1 FE modelling assumptions

The four-noded doubly curved shell element S4R with reduced integration and finite membrane strains was used throughout the numerical simulations in this chapter; this element has been successfully employed in previous numerical studies (Ma et al., 2017b; Yang et al., 2018) regarding the modelling of welded I-section structural elements. The cross-sectional mesh size was taken as the minimum of $(B+H)/40$ and the flange thickness t_f , where B is the flange width and H is the outer section depth. The adopted cross-sectional mesh size was also assigned uniformly along the longitudinal direction of the modelled members so that the aspect ratio of the elements were close to unity. Weld fillets were explicitly modelled by using five web elements with equal height but varying thicknesses, as illustrated in Figure 3.13. The thicknesses of these web elements were determined in the validation study such that the cross-sectional area of each FE model was equal to that of the corresponding test specimen. This modelling technique has also been employed in the previous studies on hot-rolled (Yun et al., 2018c) and welded I-sections (Liu et al., 2022a, 2022b). Note that, for simplicity, weld fillets are not considered in the parametric studies.

The symmetric nature of the beam-column specimens enabled only half of each member length to be modelled, with longitudinal symmetry boundary conditions applied at mid-length, to improve the computational efficiency, as shown in Figure 5.1. For ease of application of boundary conditions, the nodes at the end section of the FE model were coupled to a reference

point by means of kinematic coupling, where pinned in-plane and fixed out-of-plane boundary conditions were simulated in accordance with the experiments. The reference point was offset from the centre of the end section longitudinally by a distance of $e_{0,x}$ to represent the distance between the specimen end and the tip of the knife-edge, and laterally by a distance of $e_{0,y}$ (or $e_{0,z}$) to consider the applied loading eccentricity in the major axis (or minor axis) direction according to the experiments, as shown in Figure 5.1. The engineering stress-strain curves, measured from the tensile coupon tests in (Ma et al., 2017b; Yang et al., 2018), were converted into true stress-logarithmic plastic strain curves before inputting into ABAQUS.

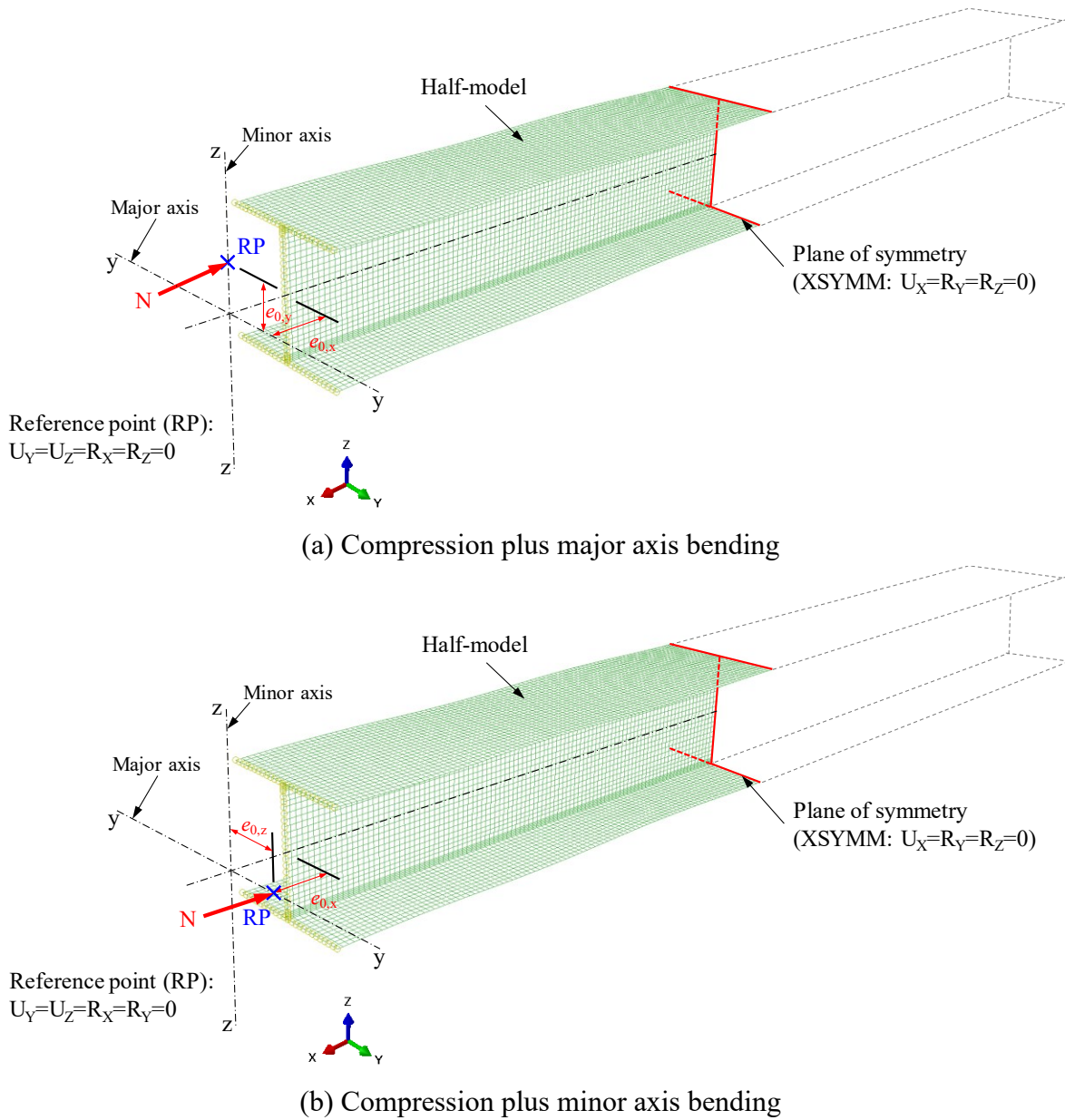


Figure 5.1: Loading and boundary conditions of FE models of welded I-section beam-columns

Both global and local geometric imperfections were introduced into the FE models by adjusting initial nodal coordinates of the perfect geometry; the modelling assumptions and techniques have been described in Section 3.4.2. Note that the determined elastic local buckling half-wavelength $L_{b,cs}$ was modified in order to fit an odd number of half-waves within the full member length, ensuring that the maximum local imperfection amplitudes were always located at the mid-length section where the symmetry boundary conditions were defined, as shown in Figure 5.2.

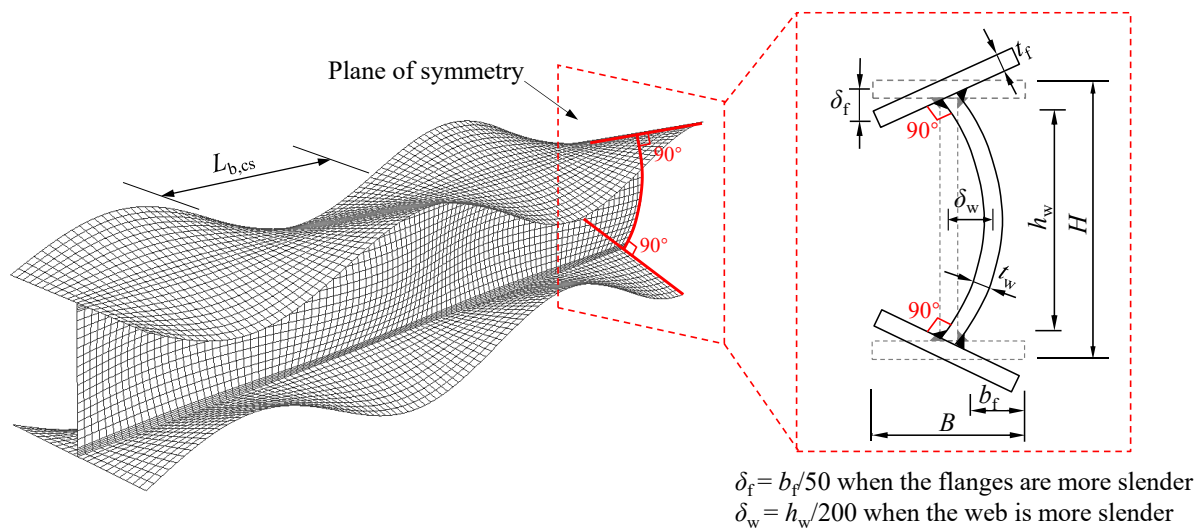


Figure 5.2: Shape and amplitudes of local geometric imperfections employed in FE models.

Residual stresses were introduced into the FE models using the residual stress pattern proposed in Section 3.2 by means of the ABAQUS *INITIAL CONDITIONS command. It was suggested that the residual stress pattern with mean values of $\sigma_{r,ft}$ and $\sigma_{r,wt}$ obtained using Equation (3.1) should be used for the validation of FE models, while the residual stress pattern with upper characteristic (95th percentile) values of $\sigma_{r,ft}$ and $\sigma_{r,wt}$ determined from Equation (3.2) should be adopted for parametric studies. Considering both material and geometrical nonlinearities, the modified Riks method was employed in order to capture the full load-deformation histories of the beam-column specimens.

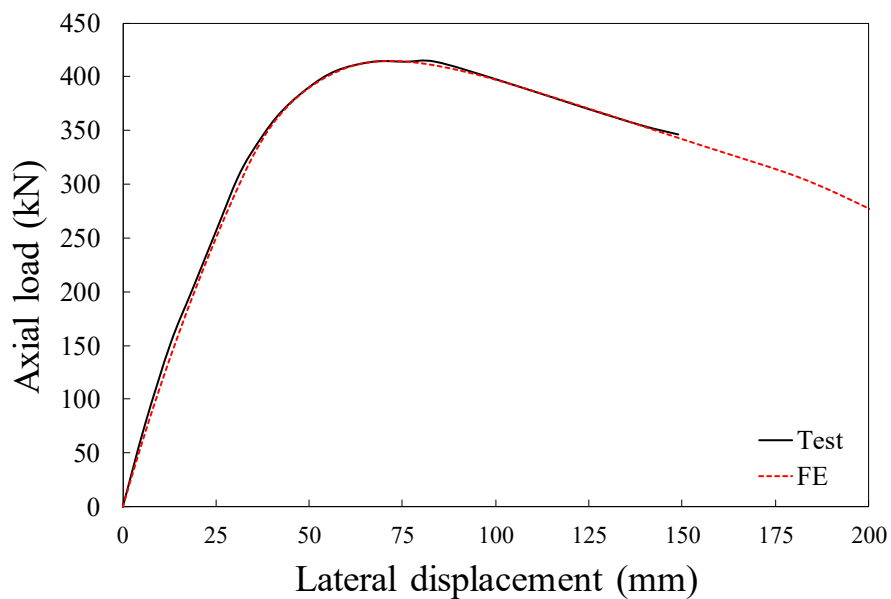
5.2.2 Validation of developed FE models

The developed FE models were validated by comparing the derived numerical predictions with the key experimental results obtained from a total of sixteen tests on welded I-section beam-columns under compression plus minor axis bending (Ma et al., 2017b; Yang et al., 2018), including the axial load-lateral displacement histories, ultimate loads and failure modes. The comparisons between the ultimate loads derived from the FE simulations $N_{u,FE}$ and those obtained from the tests $N_{u,test}$ considering varying global imperfection amplitudes are summarised in Table 5.1, where the corresponding mean values and coefficients of variation (COV) of the ratios of $N_{u,FE}/N_{u,test}$ are also provided. The comparisons in Table 5.1 reveal that the load carrying capacities of welded I-section beam-columns are generally insensitive to the variation in global imperfection amplitudes (ω_g), and the test ultimate loads $N_{u,test}$ can be generally well predicted by the developed FE models for all four investigated global imperfection amplitudes. The FE models with the global imperfection amplitude ω_g taken as $L_e/1000$ (where L_e is the distance between the knife edges at the member ends), which is the level of global imperfection assumed in the development of the column buckling design rules in EC3 (EN 1993-1-1, 2005, 2019; EN 1993-1-12, 2007), were found to predict accurate, yet slightly conservative load carrying capacities since both the global and local imperfection amplitudes adopted in the FE models (i.e. the fabrication tolerance-based values (EN 1993-1-5, 2006) shown in Figure 5.2) are generally higher than those measured in the experiments. Comparisons of the experimentally and numerically obtained axial load-lateral displacement curves for typical tested beam-columns are shown in Figure 5.3, demonstrating that the full structural response of the members can be replicated well by the FE models. Failure modes obtained from the FE models were also found to be in good agreement with the physical failure modes observed in the corresponding tested beam-column specimens, as shown in Figure 5.4.

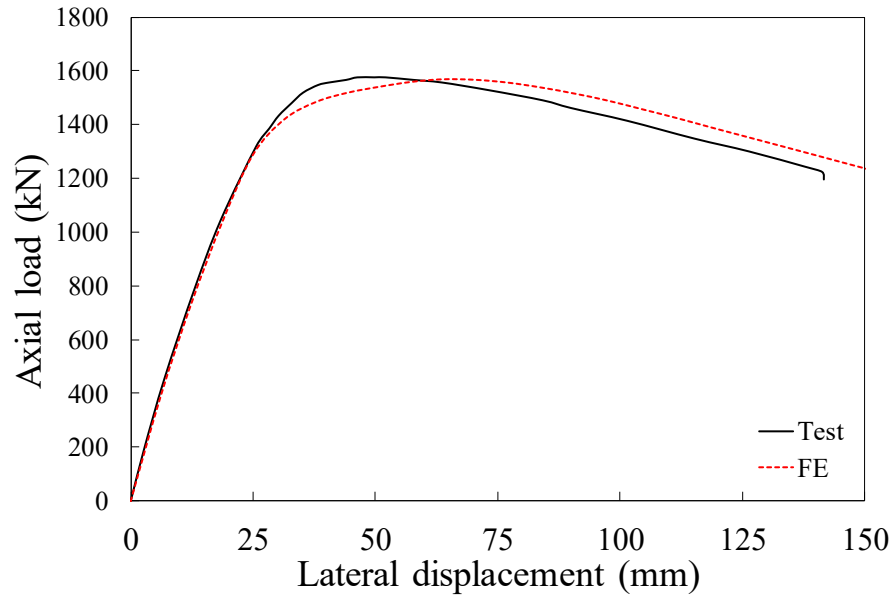
In general, the developed FE models may be seen to be able to provide accurate simulations of the structural behaviour of welded I-section beam-columns under compression plus minor axis bending and are thus deemed suitable for use in the following parametric studies.

Table 5.1: Comparisons of test and FE ultimate loads with varying levels of global imperfections.

Steel grade	Specimen label	$N_{u,FE}/N_{u,test}$			
		Global imperfection amplitude ω_g			
		$L_e/1000$	$L_e/1500$	$L_e/2000$	Measured value
Q460GJ (Yang et al., 2018)	H-314-30	0.962	0.969	0.973	0.981
	H-314-60	0.991	0.998	1.001	1.008
	H-266-45	0.983	0.991	0.995	1.005
	H-266-75	0.991	0.998	1.001	1.009
	H-218-30	0.909	0.919	0.924	0.934
	H-218-60	0.990	0.998	1.002	1.002
	H-170-45	1.014	1.023	1.027	1.031
	H-170-75	1.077	1.084	1.087	1.091
	Mean	0.990	0.997	1.001	1.008
	COV	0.048	0.046	0.046	0.044
S690 steel (Ma et al., 2017b)	EH1P	0.941	0.945	0.946	0.950
	EH1Q	1.009	1.014	1.016	1.020
	EH2P	0.940	0.943	0.945	0.949
	EH2Q	0.991	0.996	0.998	1.002
	EH3P	0.977	0.980	0.982	0.988
	EH3Q	0.961	0.965	0.967	0.969
	EH4P	0.962	0.966	0.968	0.970
	EH4Q	0.954	0.959	0.961	0.967
	Mean	0.967	0.971	0.973	0.977
	COV	0.025	0.025	0.025	0.025

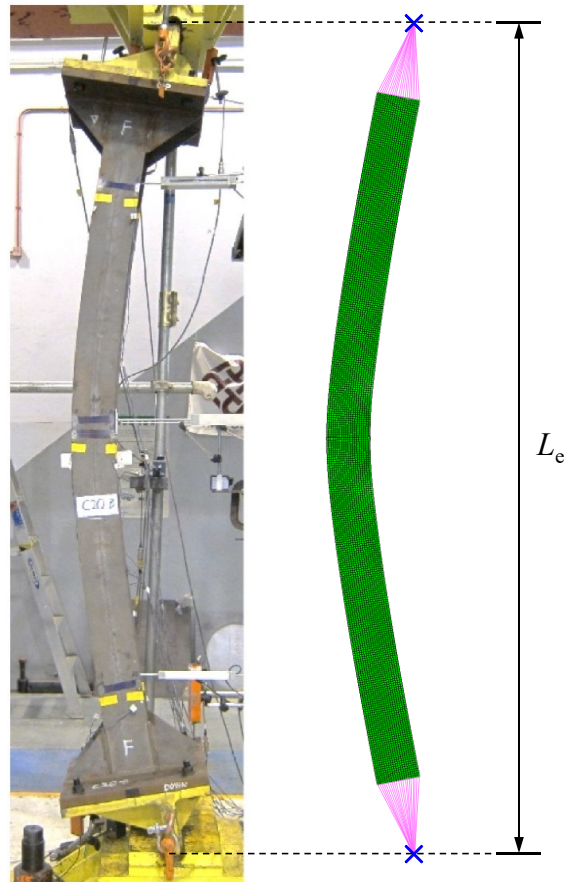


(a) Q690 steel welded I-section beam-column (Specimen EH2Q (Ma et al., 2017b))

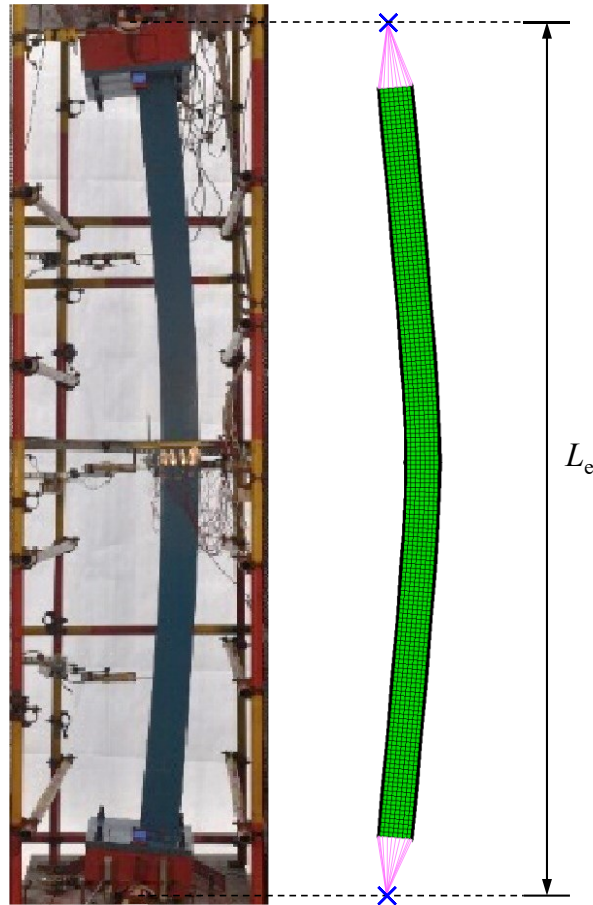


(b) Q460GJ steel welded I-section beam-column (Specimen H-218-60 (Yang et al., 2018))

Figure 5.3: Comparisons of typical test and FE load-deformation histories for welded I-section beam-columns under compression plus bending about the minor axis.



(a) Q690 steel welded I-section beam-column (Specimen EH2Q (Ma et al., 2017b))



(b) Q460GJ steel welded I-section beam-column (Specimen H-218-60 (Yang et al., 2018))

Figure 5.4: Comparisons of typical test and FE failure modes for welded I-section beam-columns under compression plus bending about the minor axis.

5.2.3 Numerical parametric studies

Upon validation of the FE models, parametric studies were conducted to generate extensive structural performance data on NSS and HSS non-slender welded I-section beam-columns with varying steel grades, cross-section slenderness, member slenderness and loading combinations. A total of five steel grades were considered in the parametric studies – S235, S355, S460, S690 and S960. The bilinear plus nonlinear hardening material model developed by Yun and Gardner (2017) was employed to generate the full-range stress-strain curves for the S235, S355, S460 and S690 steels, while the average measured stress-strain curve obtained from longitudinal tensile coupon tests in ABAQUS (2021) was used for the S960 steel throughout the parametric studies. The measured stress-strain curve of the S960 steel was employed since the scope of the bilinear plus nonlinear hardening material model developed in (Yun & Gardner, 2017) is

limited to steel grades up to S690. The basic material properties of the five investigated steel grades, including the Young's modulus E , the yield strength f_y , the ultimate tensile strength f_u and the ultimate strain ε_u corresponding to f_u are given in Table 5.2, and the full-range stress-strain curves of the five steel grades are displayed in Figure 5.5. Note that the full-range stress-strain curves shown in Figure 5.5 are all engineering stress-strains curves, which were transformed into true stress-logarithmic plastic strain curves prior to inputting into ABAQUS.

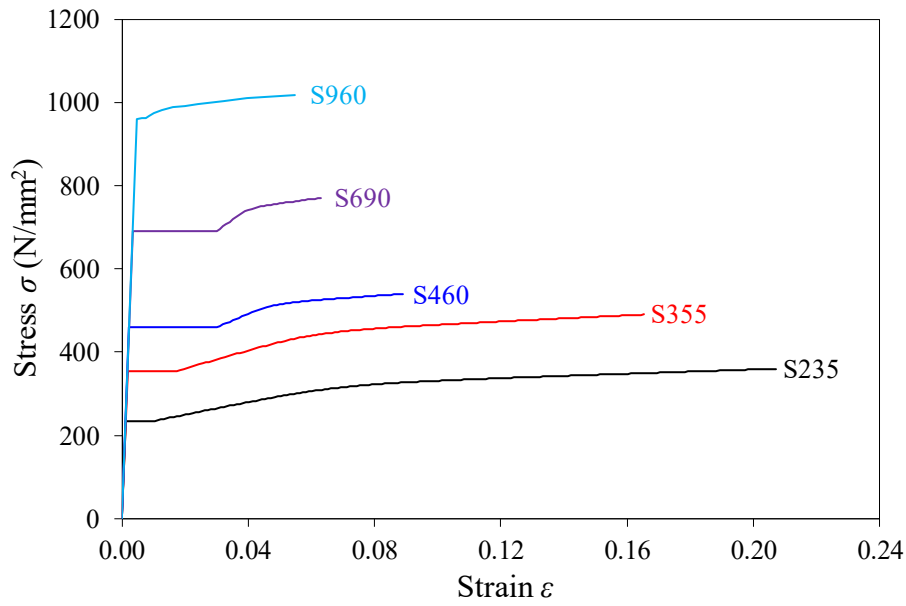


Figure 5.5: Full-range stress-strain curves for five different steel grades adopted in parametric studies.

Table 5.2: Basic material properties for five different steel grades adopted in parametric studies.

Steel grade	E N/mm ²	f_y N/mm ²	f_u N/mm ²
S235	210000	235	360
S355	210000	355	490
S460	210000	460	540
S690	210000	690	770
S960	204393	969	1024

In the parametric studies, the flange width B was set equal to 100 mm, while three different outer section depths H were considered (i.e. 100 mm, 150 mm and 200 mm), leading to three

cross-section aspect ratios H/B of 1.0, 1.5 and 2.0. For each cross-section aspect ratio, three different values of flange thickness t_f and web thickness t_w were selected to generate three different welded I-sections that covered all three non-slender cross-section classes (i.e. Class 1-3) according to the slenderness limits set out in prEN 1993-1-1 (2019) and EN 1993-1-12 (2007). For each modelled cross-section, the thicknesses of the flanges t_f and the web t_w were chosen such that the flange plate slenderness $\bar{\lambda}_{p,f}$ and the web plate slenderness $\bar{\lambda}_{p,w}$ in compression, determined in accordance with EN 1993-1-5 (2006), were approximately equal, thus minimising the effects of element interaction between the flange and the web (Gardner et al., 2019). Note that the weld fillets at each web-flange junction of the modelled I-sections were ignored in the parametric studies for modelling convenience. For each considered cross-section and for each axis of buckling, beam-column FE models with nine different lengths were selected to generate a wide spectrum of non-dimensional flexural buckling slenderness values $\bar{\lambda}$ from 0.2 to 2.6. For each beam-column FE model, a total of nine different initial loading eccentricities e_0 ranging between 1.1 mm and 4754 mm were considered to achieve a wide range of loading combinations (i.e. axial loading-to-bending moment ratios). For the beam-column FE models under compression plus major axis bending, out-of-plane lateral displacements were restrained at the web-flange intersections at regular intervals in order to suppress the flexural-torsional buckling effects. Throughout the parametric studies, the initial global and local geometric imperfection amplitudes were taken equal to $L_e/1000$ and the fabrication tolerance-based values (see Figure 5.2) (EN 1993-1-5, 2006), respectively, while the residual stress pattern followed that shown in Figure 3.2, with the upper characteristic maximum tensile residual stresses (determined from Equation (3.2)). In total, 8290 FE data on NSS and HSS non-slender welded I-section beam-columns were generated, with half of the beam-columns subjected to compression plus major axis bending and the other half subjected to compression plus minor axis bending.

5.3 Assessment of existing design provisions for NSS and HSS welded I-section beam-columns

In this section, the current design provisions for NSS and HSS non-slender welded I-section beam-columns provided in the European design standards prEN 1993-1-1 (2019) and EN 1993-

1-12 (2007), as well as the American Specification AISC 360-16 (2016), are presented. The accuracy of each design method is assessed by comparing the numerically and experimentally obtained ultimate loads $N_{u,FE(test)}$ with those predicted by the design method $N_{u,pred}$, as reported in

Table 5.3 and Table 5.4 for beam-columns under compression plus major and minor axis bending, respectively. A ratio of $N_{u,FE(test)}/N_{u,pred}$ greater than unity indicates that the design method provides a safe-sided resistance prediction and vice versa. Note that, in the assessment, all partial safety factors have been set equal to 1.0, thus facilitating a direct comparison between the FE (or test) ultimate loads $N_{u,FE(test)}$ and the strength predictions $N_{u,pred}$ obtained using the different design approaches. A dimensionless angle parameter θ is also introduced in the assessment to describe the relationship between the axial compressive load and the bending moment, as defined in Equation (5.1) and illustrated in Figure 5.6, in which $M_{u,FE(test)}$ is the numerical or experimental first order bending moment corresponding to the ultimate load $N_{u,FE(test)}$ (i.e. $M_{u,FE(test)} = N_{u,FE(test)} \cdot e_0$), and $N_{b,Rd}$ and M_{Rd} are the design resistances for column buckling and in-plane bending, respectively, determined in accordance with the considered design method. Note that $N_{b,Rd}$ and M_{Rd} represent the end points of the design interaction curve (i.e. the bending moment-axial load interaction curve), as shown in Figure 5.6. The angle parameter θ varies between 0° to 90° , with $\theta = 0^\circ$ and $\theta = 90^\circ$ representing specimens subjected to bending alone and axial compression alone, respectively.

$$\theta = \tan^{-1} \left(\frac{N_{u,FE(test)} / N_{b,Rd}}{M_{u,FE(test)} / M_{Rd}} \right) = \tan^{-1} \left(\frac{M_{Rd}}{e_0 N_{b,Rd}} \right) \quad (5.1)$$

Table 5.3: Comparison of numerically obtained ultimate loads with those predicted using different design methods for welded I-section beam-columns under compression plus major axis bending.

Steel grade (No. of FE data)	Evaluation parameter	EC3 (2019) $N_{u,FE}/N_{u,pred,EC3}$	AISC (2016) $N_{u,FE}/N_{u,pred,AISC}$	New proposals $N_{u,FE}/N_{u,pred,prop}$
S235 (No. of FE: 729)	Mean	0.961	0.954	1.076
	COV	0.051	0.101	0.033
	Maximum	1.140	1.424	1.203
	Minimum	0.869	0.839	0.959
S355 (No. of FE: 729)	Mean	0.987	0.990	1.077
	COV	0.041	0.097	0.032
	Maximum	1.148	1.444	1.193
	Minimum	0.909	0.806	0.975
S460 (No. of FE: 729)	Mean	1.005	1.014	1.081
	COV	0.036	0.096	0.031
	Maximum	1.151	1.476	1.208
	Minimum	0.916	0.869	0.976
S690 (No. of FE: 729)	Mean	1.039	1.056	1.090
	COV	0.036	0.100	0.036
	Maximum	1.235	1.532	1.283
	Minimum	0.945	0.896	0.993
S960 (No. of FE: 729)	Mean	1.069	1.104	1.103
	COV	0.030	0.108	0.035
	Maximum	1.231	1.615	1.294
	Minimum	0.959	0.920	1.005
Overall (No. of FE: 3645)	Mean	1.012	1.024	1.085
	COV	0.054	0.113	0.035
	Maximum	1.235	1.615	1.294
	Minimum	0.869	0.806	0.959

Table 5.4: Comparison of numerically and experimentally obtained ultimate loads with those predicted using different design methods for welded I-section beam-columns under compression plus minor axis bending.

Steel grade (No. of FE/test data)	Evaluation parameter	EC3 (2019) $N_{u,FE(test)}/N_{u,pred,EC3}$	AISC (2016) $N_{u,FE(test)}/N_{u,pred,AISC}$	New proposals $N_{u,FE(test)}/N_{u,pred,prop}$
S235 (No. of FE: 729)	Mean	1.007	0.884	1.119
	COV	0.055	0.118	0.045
	Maximum	1.236	1.255	1.252
	Minimum	0.890	0.736	0.971
S355 (No. of FE: 729)	Mean	1.031	0.912	1.108
	COV	0.057	0.119	0.047
	Maximum	1.282	1.304	1.230
	Minimum	0.916	0.773	0.957
S460 (No. of FE: 729 No. of tests: 14)	Mean	1.051	0.932	1.113
	COV	0.056	0.117	0.047
	Maximum	1.283	1.308	1.267
	Minimum	0.923	0.779	0.977
S690 (No. of FE: 729 No. of tests: 4)	Mean	1.078	0.965	1.113
	COV	0.055	0.115	0.049
	Maximum	1.288	1.308	1.322
	Minimum	0.936	0.787	0.984
S960 (No. of FE: 729)	Mean	1.114	1.004	1.120
	COV	0.056	0.113	0.050
	Maximum	1.332	1.346	1.223
	Minimum	0.952	0.805	0.973
Overall (No. of FE: 3645 No. of tests: 18)	Mean	1.056	0.940	1.115
	COV	0.066	0.125	0.048
	Maximum	1.332	1.346	1.322
	Minimum	0.890	0.736	0.957

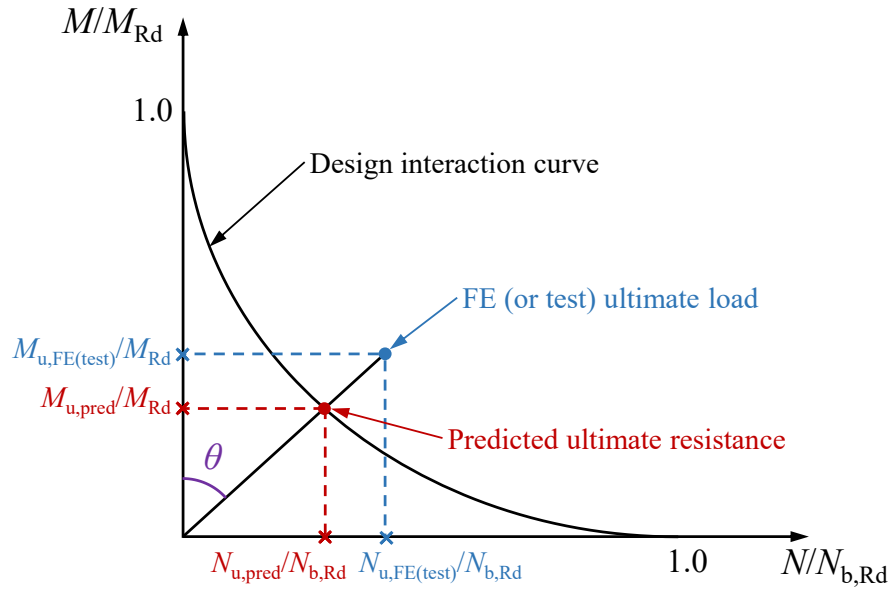


Figure 5.6: Definition of the angle parameter θ on moment-axial compression (M-N) interaction curve.

5.3.1 European Standards prEN 1993-1-1 (2019) and EN 1993-1-12 (2007) (EC3)

According to the European design standards prEN 1993-1-1 (2019) and EN 1993-1-12 (2007), Equations (5.2) and (5.3) are given for the in-plane stability design of beam-columns with Class 1-3 welded I-sections under compression plus major axis bending and minor axis bending, respectively,

$$\frac{N_{Ed}}{N_{b,y,EC3}} + k_{yy,EC3} \frac{M_{y,Ed}}{M_{y,EC3}} \leq 1.0 \quad (5.2)$$

$$\frac{N_{Ed}}{N_{b,z,EC3}} + k_{zz,EC3} \frac{M_{z,Ed}}{M_{z,EC3}} \leq 1.0 \quad (5.3)$$

where N_{Ed} is the design axial compressive load, $M_{y,Ed}$ and $M_{z,Ed}$ are the design first order bending moments about the major and minor axes, respectively, $N_{b,y,EC3}$ and $N_{b,z,EC3}$ are the EC3 column flexural buckling resistances about the major and minor axes, respectively, $M_{y,EC3}$ and $M_{z,EC3}$ are the EC3 cross-sectional bending resistances about the major and minor axes, respectively, and $k_{yy,EC3}$ and $k_{zz,EC3}$ are the EC3 interaction factors for beam-columns subjected

to compression plus major and minor axis bending, respectively. Specifically, the EC3 flexural buckling resistance $N_{b,EC3}$ for columns with Class 1-3 welded I-sections can be determined from:

$$N_{b,EC3} = \chi_{EC3} A f_y \quad (5.4)$$

where A is the gross cross-section area and χ_{EC3} is the EC3 flexural buckling reduction factor about the relevant buckling axis determined from Equations (3.9) and (3.10). In the equations, $\bar{\lambda}$ is the column non-dimensional slenderness given by $(A f_y / N_{cr})^{0.5}$, where N_{cr} is the elastic buckling load about the relevant buckling axis, α_{EC3} is the EC3 imperfection factor, taken equal to 0.34 and 0.49 for both NSS and HSS welded I-section columns buckling about the major and minor axes, respectively (i.e. corresponding to the EC3 column buckling curves “b” and “c”, respectively), though a recent study (Tankova et al., 2022) recommended the use of revised column buckling curves “a” and “b” for the design of HSS columns buckling about the major and minor axes, respectively (i.e. imperfection factors α_{EC3} equal to 0.21 and 0.34, respectively).

With regards to the EC3 cross-sectional bending resistances M_{EC3} , prEN 1993-1-1 (2019) prescribes the use of the plastic moment capacity M_{pl} (i.e. the product of the plastic section modulus W_{pl} and the yield strength f_y) for Class 1 and 2 welded I-sections, and the elasto-plastic moment capacity M_{ep} (i.e. the product of the elasto-plastic section modulus W_{ep} , as given by Equation (5.5), and the yield strength f_y) for Class 3 welded I-sections. In accordance with the Annex B of prEN 1993-1-1 (2019), W_{ep} is defined based on linear interpolation between W_{pl} and the elastic section modulus W_{el} with respect to the parameter β_{ep} , as given by Equation (5.6):

$$W_{ep} = W_{pl} - (W_{pl} - W_{el}) \beta_{ep} \quad (5.5)$$

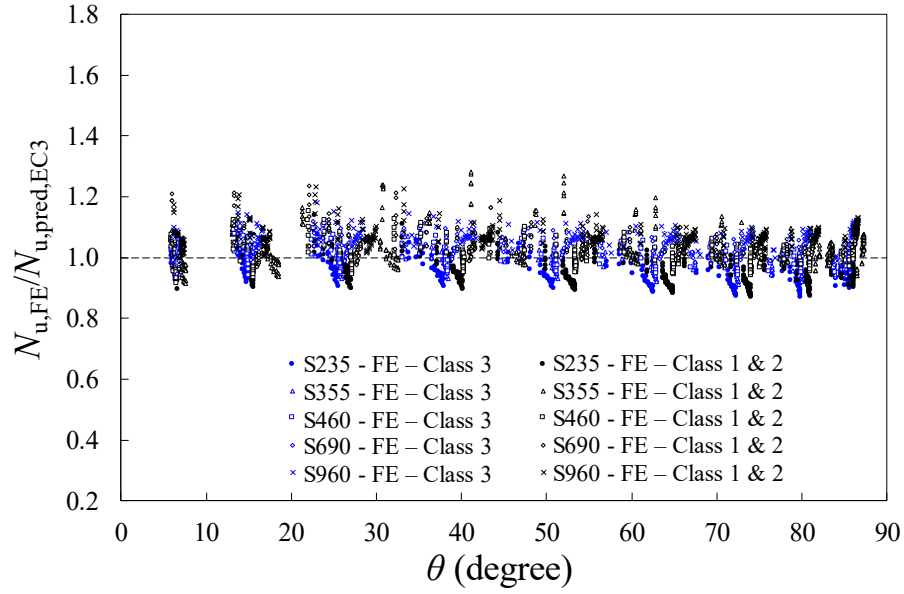
$$\beta_{ep} = \begin{cases} \max\left(\frac{b_f/t_f - 10\varepsilon}{4\varepsilon}; \frac{h_w/t_w - 83\varepsilon}{38\varepsilon}; 0\right) \leq 1.0, & \text{for bending about the major axis} \\ \max\left(\frac{b_f/t_f - 10\varepsilon}{6\varepsilon}; 0\right) \leq 1.0, & \text{for bending about the minor axis,} \end{cases} \quad (5.6)$$

where ε is the material parameter equal to $(235/f_y)^{0.5}$. The EC3 interaction factors $k_{yy,EC3}$ and $k_{zz,EC3}$ for Class 1 to 3 welded I-sections can be determined from Equations (5.7) and (5.8), respectively. In Equations (5.7) and (5.8), C_{my} and C_{mz} are the equivalent uniform moment factors about the major and minor axes, respectively, which are equal to 1.0 for the investigated loading cases of uniform bending throughout the present study.

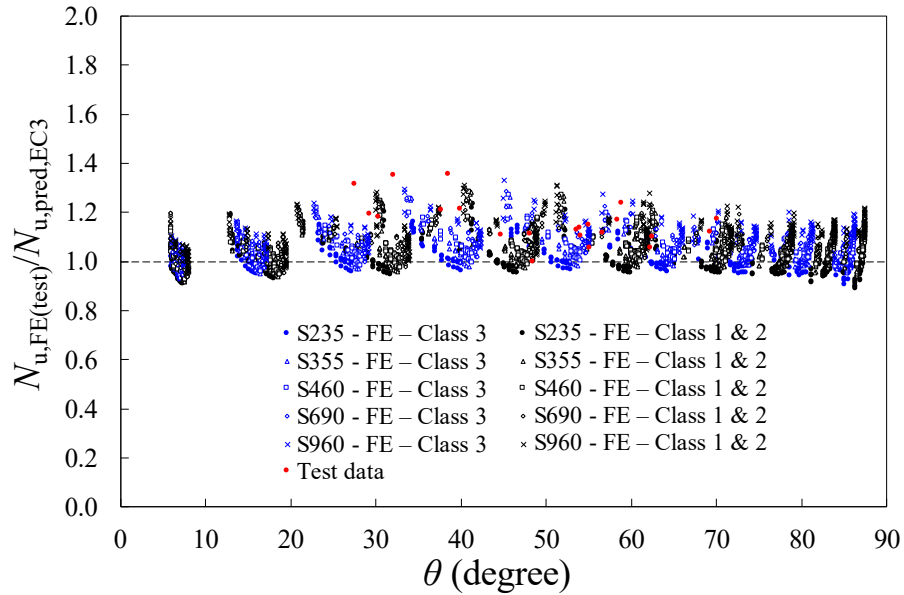
$$k_{yy,EC3} = \begin{cases} C_{my} \left[1 + (\bar{\lambda}_y - 0.2) \frac{N_{Ed}}{N_{b,y,EC3}} \right], & \text{for } \bar{\lambda}_y < 1.0 \\ C_{my} \left(1 + 0.8 \frac{N_{Ed}}{N_{b,y,EC3}} \right), & \text{for } \bar{\lambda}_y \geq 1.0 \end{cases} \quad (5.7)$$

$$k_{zz,EC3} = \begin{cases} C_{mz} \left[1 + (2\bar{\lambda}_z - 0.6) \frac{N_{Ed}}{N_{b,z,EC3}} \right], & \text{for } \bar{\lambda}_z < 1.0 \\ C_{mz} \left(1 + 1.4 \frac{N_{Ed}}{N_{b,z,EC3}} \right), & \text{for } \bar{\lambda}_z \geq 1.0 \end{cases} \quad (5.8)$$

The ratios of the ultimate loads obtained from the FE simulations and tests $N_{u,FE(test)}$ to the EC3 ultimate resistance predictions $N_{u,pred,EC3}$ are plotted against the corresponding angle parameter θ in Figure 5.7(a) and (b) to investigate the accuracy of the EC3 design method for welded I-section beam-columns under compression plus major and minor axis bending, respectively. The overall mean and COV values of the load ratios $N_{u,FE(test)}/N_{u,pred,EC3}$ are equal to 1.012 and 0.054 for beam-columns under compression plus major axis bending, and equal to 1.056 and 0.066 for compression plus minor axis bending, as listed in Table 5.3 and Table 5.4. It can be concluded from these statistical results and Figure 5.7(a) and (b) that the current EC3 design method generally yields reasonable, but somewhat scattered ultimate resistance predictions for NSS and HSS non-slender welded I-section beam-columns, especially for those under compression plus minor axis bending. However, there are a set of data points of beam-columns under compression plus major axis bending lying on the unsafe side. The statistical results in Table 5.3 and Table 5.4 also suggest that there is generally an increasing level of conservatism with increasing steel grade; this is due primarily to the neglect of the reducing relative influence of residual stresses on the ultimate resistance of beam-columns with increasing steel grades (Tse et al., 2021; Yun et al., 2022).



(a) Compression plus major axis bending



(b) Compression plus minor axis bending

Figure 5.7: Comparison of FE and test results with EC3 resistance predictions for NSS and HSS non-slender welded I-section beam-columns under combined loading.

5.3.2 American Specification AISC 360-16 (2016) (AISC)

Equations (5.9) and (5.10) are given in the American Specification AISC 360-16 (2016) (AISC) for the design of non-slender welded I-section beam-columns subjected to compression plus major and minor axis bending, respectively, where $N_{b,y,AISC}$ and $N_{b,z,AISC}$ are the AISC column flexural buckling resistances about the major and minor axes, respectively, $M_{y,AISC}$ and $M_{z,AISC}$

are the AISC cross-sectional bending resistances about the major and minor axes, respectively, and α_{amp} is an amplification factor that accounts for the second-order effects in beam-column members, as given by Equation (5.11).

$$\begin{cases} \frac{N_{\text{Ed}}}{N_{\text{b,y,AISC}}} + \frac{8}{9} \frac{\alpha_{\text{amp}} M_{\text{y,Ed}}}{M_{\text{y,AISC}}} \leq 1.0, & \text{if } \frac{N_{\text{Ed}}}{N_{\text{b,y,AISC}}} \geq 0.2, \\ \frac{N_{\text{Ed}}}{2N_{\text{b,y,AISC}}} + \frac{\alpha_{\text{amp}} M_{\text{y,Ed}}}{M_{\text{y,AISC}}} \leq 1.0, & \text{if } \frac{N_{\text{Ed}}}{N_{\text{b,y,AISC}}} < 0.2, \end{cases} \quad (5.9)$$

$$\begin{cases} \frac{N_{\text{Ed}}}{N_{\text{b,z,AISC}}} + \frac{8}{9} \frac{\alpha_{\text{amp}} M_{\text{z,Ed}}}{M_{\text{z,AISC}}} \leq 1.0, & \text{if } \frac{N_{\text{Ed}}}{N_{\text{b,z,AISC}}} \geq 0.2, \\ \frac{N_{\text{Ed}}}{2N_{\text{b,z,AISC}}} + \frac{\alpha_{\text{amp}} M_{\text{z,Ed}}}{M_{\text{z,AISC}}} \leq 1.0, & \text{if } \frac{N_{\text{Ed}}}{N_{\text{b,z,AISC}}} < 0.2, \end{cases} \quad (5.10)$$

$$\alpha_{\text{amp}} = \frac{1}{1 - (N_{\text{Ed}}/N_{\text{cr}})} \quad (5.11)$$

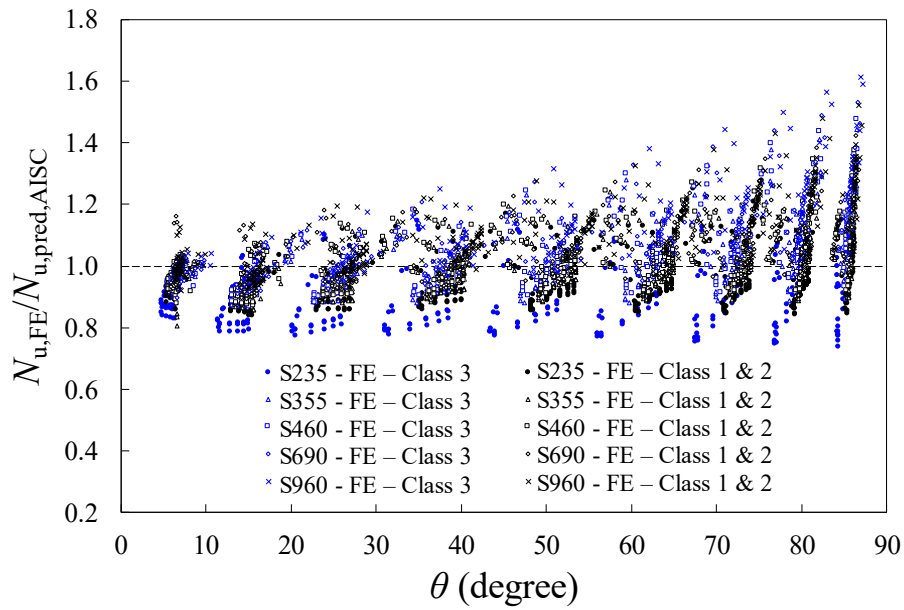
The AISC flexural buckling resistance $N_{\text{b,AISC}}$ for columns with non-slender welded I-sections can be expressed by Equation (5.12), where χ_{AISC} is the AISC flexural buckling reduction factor about the relevant buckling axis determined from Equation (3.12).

$$N_{\text{b,AISC}} = \chi_{\text{AISC}} A f_y \quad (5.12)$$

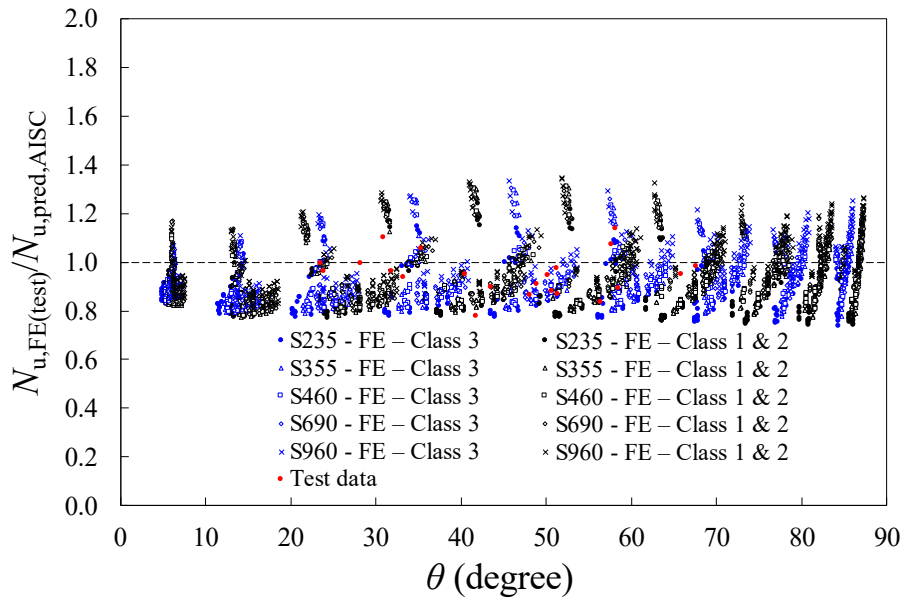
The AISC cross-sectional bending resistance M_{AISC} for compact cross-sections is the same as that for Class 1 and 2 cross-sections in EC3 (2019) i.e. the plastic moment resistance M_{pl} , while an inelastic reserve method is used for noncompact cross-sections, which accounts for the effect of the partial spread of plasticity, as detailed in Section F of AISC 360-16 (2016).

To assess the accuracy of the AISC beam-column design provisions, the FE and test ultimate loads $N_{\text{u,FE(test)}}$ are normalised by the AISC predicted ultimate resistances $N_{\text{u,pred,AISC}}$ and plotted against the corresponding angle parameter θ in Figure 5.8(a) and (b) for the members under compression plus major and minor axis bending, respectively. The overall mean and COV

values of the load ratios $N_{u,FE(test)}/N_{u,pred,AISC}$ are equal to 1.024 and 0.113 for beam-columns under compression plus major axis bending, and equal to 0.94 and 0.125 for compression plus minor axis bending, as provided in Table 5.3 and Table 5.4. As can be seen from Figure 5.8 and Table 5.3 and Table 5.4, the AISC design method offers very scattered ultimate resistance predictions for both NSS and HSS non-slender welded I-section beam-columns, with a large number of data points falling on the unsafe side; this is attributed largely to the interaction curves adopted in AISC 360-16 (2016).



(a) Compression plus major axis bending



(b) Compression plus minor axis bending

Figure 5.8: Comparison of FE and test results with AISC resistance predictions for NSS and HSS non-slender welded I-section beam-columns under combined loading.

5.4 New design proposals

The assessment presented in the previous section reveals that the current EC3 and AISC design methods provide somewhat inaccurate and scattered ultimate resistance predictions for NSS and HSS non-slender welded I-section beam-columns under compression plus uniaxial bending. Hence, new design proposals are set out in this section with the aim to achieve improved accuracy in the design of NSS and HSS non-slender welded I-section beam-columns. The new design proposals adopt the general format of the design interaction formulae specified in EN 1993-1-1 (2005, 2019), with improvements made by employing more accurate end points (i.e. column flexural buckling strengths and cross-sectional bending resistances) and updated interaction factors.

5.4.1 New design formulae

The proposed design formulae are given in Equations (5.13) and (5.14) for NSS and HSS non-slender welded I-section beam-columns under compression plus major and minor axis bending, respectively.

$$\frac{N_{Ed}}{N_{b,y,mod-EC3}/\gamma_{M1}} + k_{yy,prop} \frac{M_{y,Ed}}{M_{y,csn}/\gamma_{M1}} \leq 1.0 \quad (5.13)$$

$$\frac{N_{Ed}}{N_{b,z,mod-EC3}/\gamma_{M1}} + k_{zz,prop} \frac{M_{z,Ed}}{M_{z,csn}/\gamma_{M1}} \leq 1.0 \quad (5.14)$$

In Equations (5.13) and (5.14), $N_{b,y,mod-EC3}$ and $N_{b,z,mod-EC3}$ are the column flexural buckling resistances about the major and minor axes, respectively, determined using the modified EC3 column flexural buckling curves proposed in Chapter 3. The modified EC3 flexural buckling resistances $N_{b,mod-EC3}$ for columns with Class 1-3 welded I-sections can be calculated using:

$$N_{b,mod-EC3} = \chi_{mod-EC3} A f_y \quad (5.15)$$

where $\chi_{mod-EC3}$ is the modified EC3 flexural buckling reduction factor about the relevant buckling axis determined from Equations (3.13)-(3.16).

As discussed in Chapter 3, the modified EC3 method has been shown to provide more accurate and consistent predictions of flexural buckling resistances for both NSS and HSS non-slender welded I-section columns than the current EC3 and AISC provisions, since the influence of yield strength on the flexural buckling resistances of welded I-section columns is rationally considered by using the modified imperfection factor $\alpha_{\text{mod-EC3}}$. The modified EC3 method is therefore used to determine the compression end point (i.e. the column flexural buckling resistance) of the new design proposals for welded I-section beam-columns.

The bending end point of the new beam-column design proposals (i.e. the cross-sectional bending resistance about the major axis $M_{y,\text{csm}}$ in Equations (5.13) or the minor axis $M_{z,\text{csm}}$ in Equations (5.14)) is calculated by means of the Continuous Strength Method (CSM) (Gardner, 2008; Gardner et al., 2011, 2023), which enables a rational allowance to be made for material nonlinearity (i.e. the spread of plasticity and strain hardening) in the determination of cross-sectional resistances (Yun et al., 2018a; Yun & Gardner, 2018). The CSM cross-sectional resistances for I-sections in major axis bending $M_{y,\text{csm}}$ and minor axis bending $M_{z,\text{csm}}$ are given by Equations (5.16) and (5.17), respectively, where ε_y is the material yield strain equal to f_y/E , and ε_{sh} and E_{sh} are the strain hardening strain and strain hardening modulus from the employed quad-linear material model, as detailed in Yun and Gardner (2017).

$$M_{y,\text{csm}} = \begin{cases} W_{\text{pl},y} f_y \left[1 - \left(1 - \frac{W_{\text{el},y}}{W_{\text{pl},y}} \right) / \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \right)^2 \right], & \text{for } \varepsilon_{\text{csm}} \leq \varepsilon_{\text{sh}} \\ W_{\text{pl},y} f_y \left[1 - \left(1 - \frac{W_{\text{el},y}}{W_{\text{pl},y}} \right) / \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \right)^2 + 0.08 \frac{E_{\text{sh}}}{E} \left(\frac{\varepsilon_{\text{csm}} - \varepsilon_{\text{sh}}}{\varepsilon_y} \right)^2 \right], & \text{for } \varepsilon_{\text{csm}} > \varepsilon_{\text{sh}} \end{cases} \quad (5.16)$$

$$M_{z,\text{csm}} = \begin{cases} W_{\text{pl},z} f_y \left[1 - \left(1 - \frac{W_{\text{el},z}}{W_{\text{pl},z}} \right) / \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \right)^{1.2} \right], & \text{for } \varepsilon_{\text{csm}} \leq \varepsilon_{\text{sh}} \\ W_{\text{pl},z} f_y \left[1 - \left(1 - \frac{W_{\text{el},z}}{W_{\text{pl},z}} \right) / \left(\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} \right)^{1.2} + 0.05 \frac{E_{\text{sh}}}{E} \left(\frac{\varepsilon_{\text{csm}} - \varepsilon_{\text{sh}}}{\varepsilon_y} \right)^2 \right], & \text{for } \varepsilon_{\text{csm}} > \varepsilon_{\text{sh}} \end{cases} \quad (5.17)$$

In Equations (5.16) and (5.17), ε_{csm} is the limiting CSM strain that defines the maximum strain that a cross-section can endure prior to failure, defined by Equations (2.1) and (2.2). The upper bound of Ω that sets the maximum tolerable level of plastic strain was taken equal to 15 in this study, which is a generally recommended value in line with the EN 1993-1-1 (2005) ductility requirement. In Equation (2.2), the elastic local buckling stress of the welded I-section in bending about the relevant axis $\sigma_{\text{cr,cs}}$ is obtained through numerical methods, such as the finite strip software CUFSM (Schafer & Adany, 2006), or alternatively through the analytical formulae developed by Gardner et al. (Gardner et al., 2019). The CSM has been shown to yield more accurate and consistent bending moment capacity predictions for I-section profiles compared to the current EC3 and AISC design provisions (Lan et al., 2019; Yun et al., 2018a). The CSM is thus employed in the new beam-column design proposals to determine the bending end point of the interaction curves (i.e. Equations (5.16) and (5.17)). Based on the improved compression and bending end points, new interaction factors (i.e. $k_{yy,\text{prop}}$ and $k_{zz,\text{prop}}$ for the major and minor axis bending, respectively) are now calibrated using the numerical dataset derived in Subsection 5.2.3, as described in the following subsection.

5.4.2 Calibration of new interaction factors

Data for two cross-section sizes – $100 \times 150 \times 10 \times 10$ and $100 \times 200 \times 12 \times 16$ ($B \times H \times t_f \times t_w$, in mm) with modified EC3 compressive load levels (i.e. $n_{\text{mod-EC3,y}}$ for major axis bending and $n_{\text{mod-EC3,z}}$ for minor axis bending, as defined by Equations (5.18) and (5.19), respectively) varying from 0.2 to 0.8, in increments of 0.2, were isolated from the full numerical database generated in Subsection 5.2.3 and used for the calibration of the new interaction factors $k_{yy,\text{prop}}$ and $k_{zz,\text{prop}}$.

$$n_{\text{mod-EC3,y}} = \frac{N_{\text{Ed}}}{N_{\text{b,y,mod-EC3}}} \quad (5.18)$$

$$n_{\text{mod-EC3,z}} = \frac{N_{\text{Ed}}}{N_{\text{b,z,mod-EC3}}} \quad (5.19)$$

The benchmark (target) FE interaction factors $k_{yy,FE}$ and $k_{zz,FE}$ for these considered beam-columns were back-calculated from the numerical results using Equations (5.20) and (5.21), respectively, which are rearranged forms of the newly proposed design formulae (i.e. Equations (5.13) and (5.14)).

$$k_{yy,FE} = \left(1 - n_{\text{mod-EC3},y}\right) \frac{M_{\text{csm}}}{M_{\text{Ed}}} \quad (5.20)$$

$$k_{zz,FE} = \left(1 - n_{\text{mod-EC3},z}\right) \frac{M_{\text{csm}}}{M_{\text{Ed}}} \quad (5.21)$$

The numerically derived benchmark interaction factors $k_{yy,FE}$ and $k_{zz,FE}$ are plotted against the corresponding column non-dimensional slenderness $\bar{\lambda}$ for the four different compressive load levels $n_{\text{mod-EC3}}$ of 0.2, 0.4, 0.6 and 0.8 in Figures 5.9-5.13 for beam-columns made of varying steel grades. Design formulae for the new interaction factors $k_{yy,\text{prop}}$ and $k_{zz,\text{prop}}$ are proposed based on these benchmark interaction factors $k_{yy,FE}$ and $k_{zz,FE}$, employing a bilinear form, in line with the current EC3 design approach (EN 1993-1-1, 2005, 2019), as expressed by Equations (5.22) and (5.23), respectively:

$$k_{yy,\text{prop}} = C_{\text{my}} \left[1 + D_{1,y} \left(\bar{\lambda}_y - D_{2,y} \right) n_{\text{mod-EC3},y} \right] \leq C_{\text{my}} \left(1 + D_{3,y} n_{\text{mod-EC3},y} \right) \quad (5.22)$$

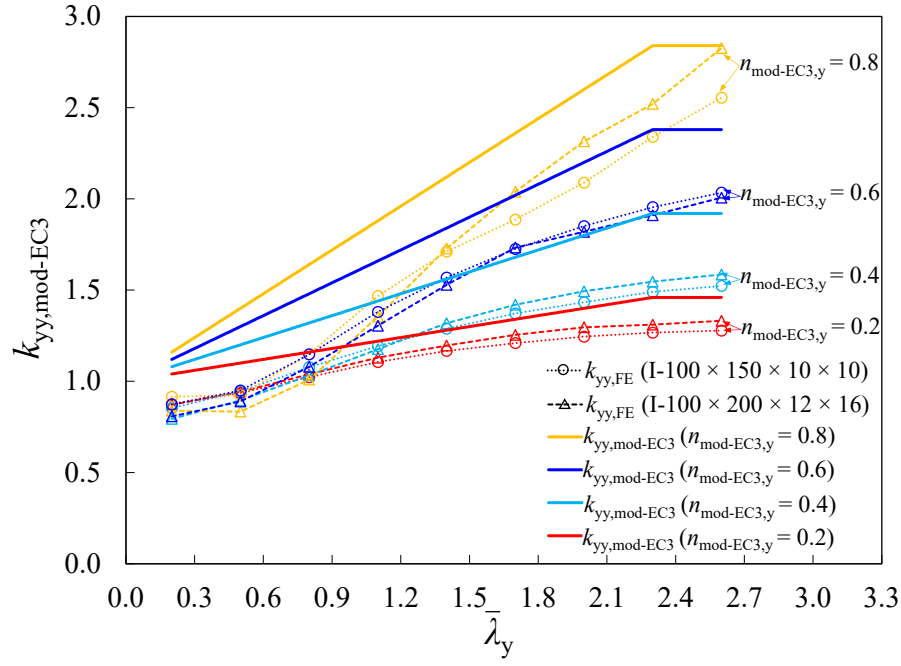
$$k_{zz,\text{prop}} = C_{\text{mz}} \left[1 + D_{1,z} \left(\bar{\lambda}_z - D_{2,z} \right) n_{\text{mod-EC3},z} \right] \leq C_{\text{mz}} \min \left(1 + D_{3,z} n_{\text{mod-EC3},z}, D_{3,z} \right) \quad (5.23)$$

where $D_{1,y}$, $D_{1,z}$, $D_{2,y}$, $D_{2,z}$, $D_{3,y}$ and $D_{3,z}$ are coefficients that are determined through calibration against the numerically obtained datasets. The proposed coefficients are yield strength-dependent in order to capture the influence of yield strength on the shape of the interaction curves of welded I-sections made of varying steel grades, as summarised in Equation (5.24) for $D_{1,y}$, $D_{2,y}$ and $D_{3,y}$ and Equation (5.25) for $D_{1,z}$, $D_{2,z}$ and $D_{3,z}$.

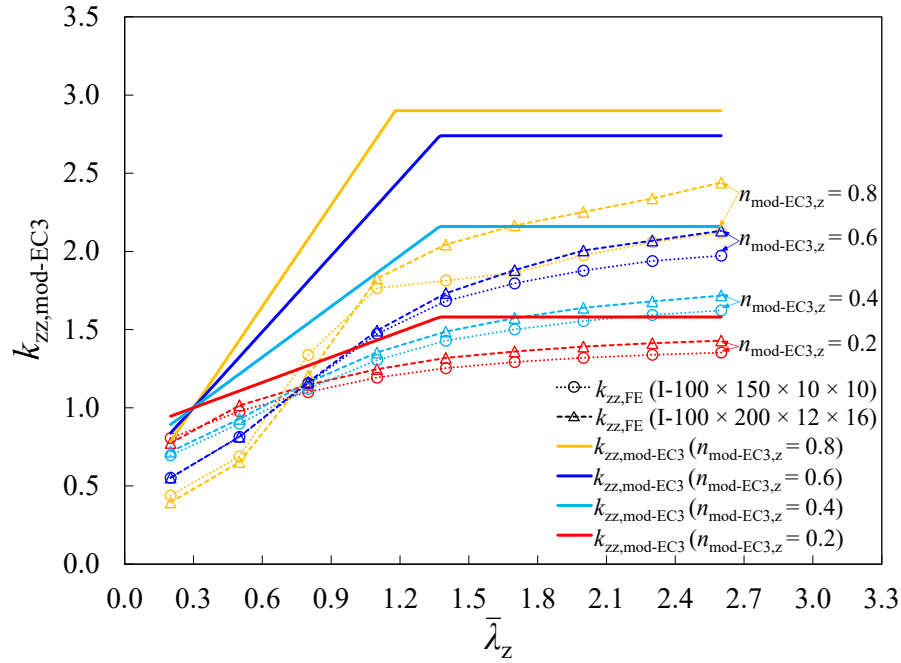
$$\begin{cases} D_{1,y} = 1 \\ D_{2,y} = 0 \\ D_{3,y} = 1.6\varepsilon + 0.7 \end{cases} \quad (5.24)$$

$$\begin{cases} D_{1,z} = 3.7 - \varepsilon \\ D_{2,z} = 0.5 - 0.2\varepsilon \\ D_{3,z} = 0.4\varepsilon + 2.5 \end{cases} \quad (5.25)$$

The proposed interaction factors $k_{yy,\text{prop}}$ and $k_{zz,\text{prop}}$ are compared with the numerically derived benchmark interaction factors $k_{yy,\text{FE}}$ and $k_{zz,\text{FE}}$ in Figures 5.9-5.13 for welded I-section beam-columns made of varying steel grades. It can be observed from Figures 5.9-5.13 that the proposed interaction factors generally follow the trend of the benchmark FE data points and offer suitably safe-sided predictions, though there are relatively large discrepancies between the proposed and FE derived interaction factors for beam-columns with large slenderness values $\bar{\lambda}$ under high axial compressive load levels $n_{\text{mod-EC3}}$. However, these discrepancies have a very small influence on the accuracy of the resistance predictions since, at high axial load levels, the axial compression term (see Equations (5.13) and (5.14)) is dominant (Bu & Gardner, 2019b; Kucukler & Gardner, 2021; Meng & Gardner, 2021; Tse et al., 2021), while the bending term, which features the interaction factor, is small.

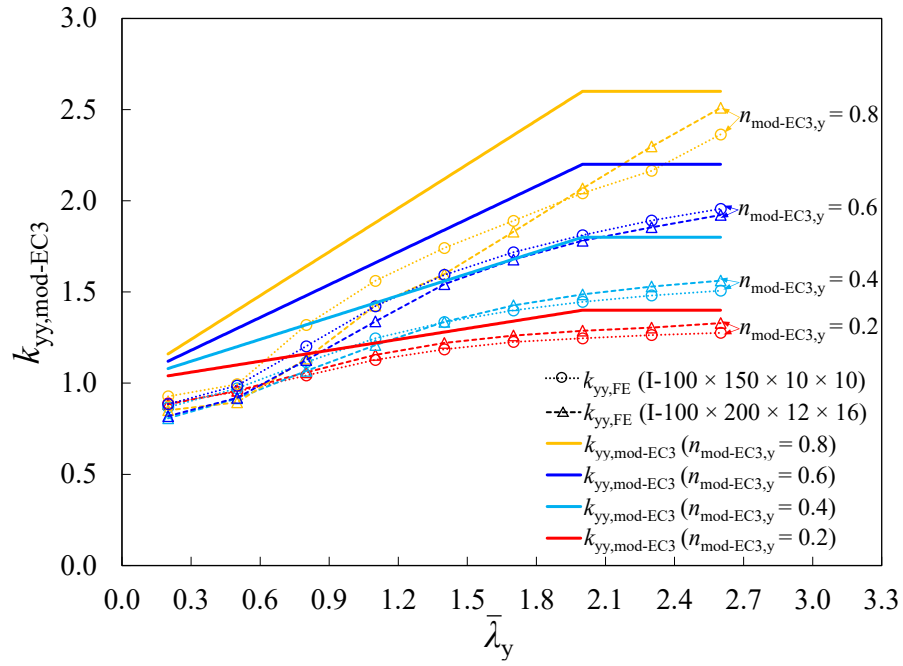


(a) Compression plus major axis bending

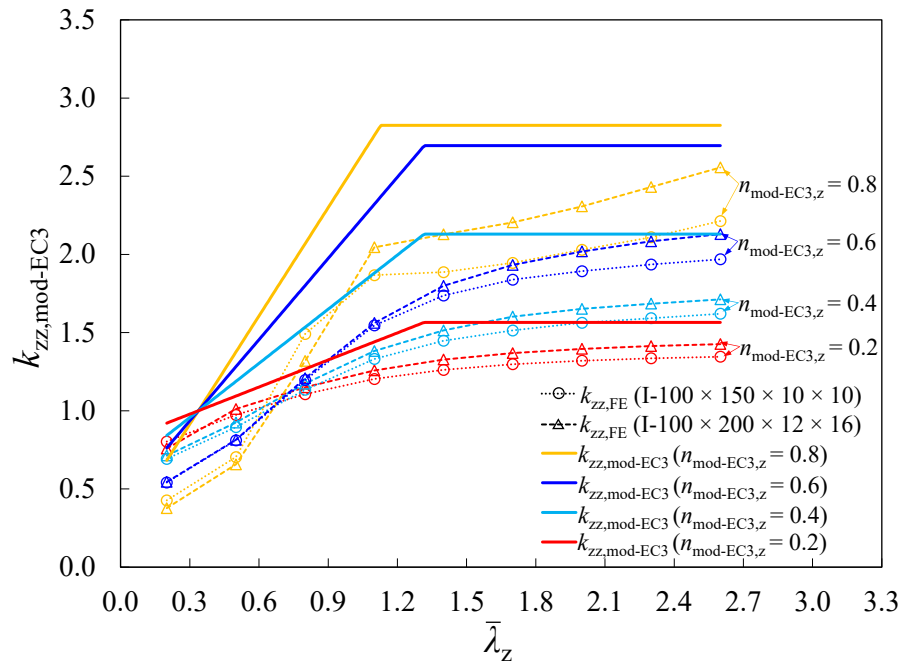


(b) Compression plus minor axis bending

Figure 5.9: Calibration of the modified interaction factors $k_{yy,\text{mod-EC3}}$ and $k_{zz,\text{mod-EC3}}$ for S235 steel welded I-section beam-columns.

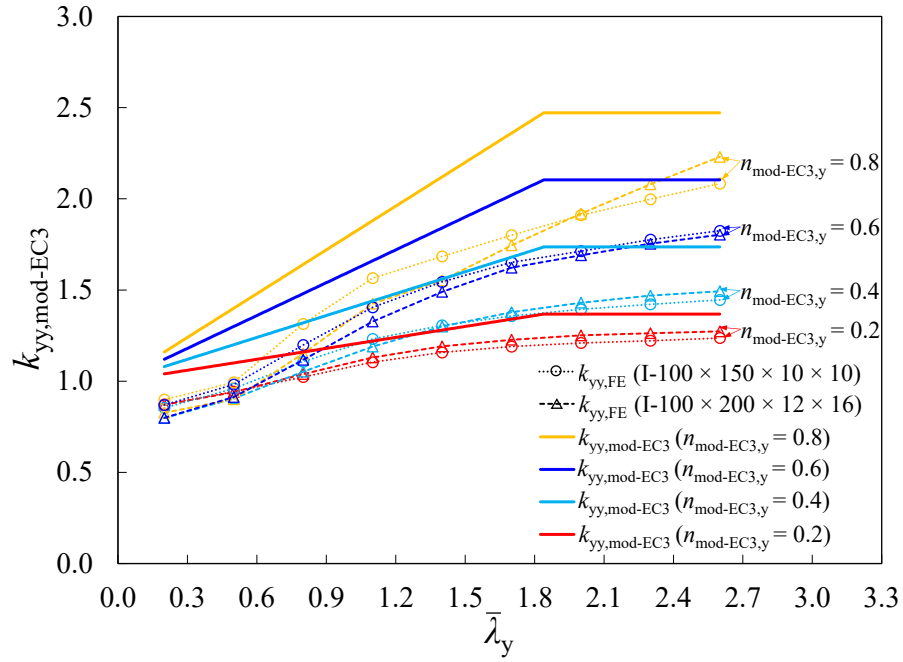


(a) Compression plus major axis bending

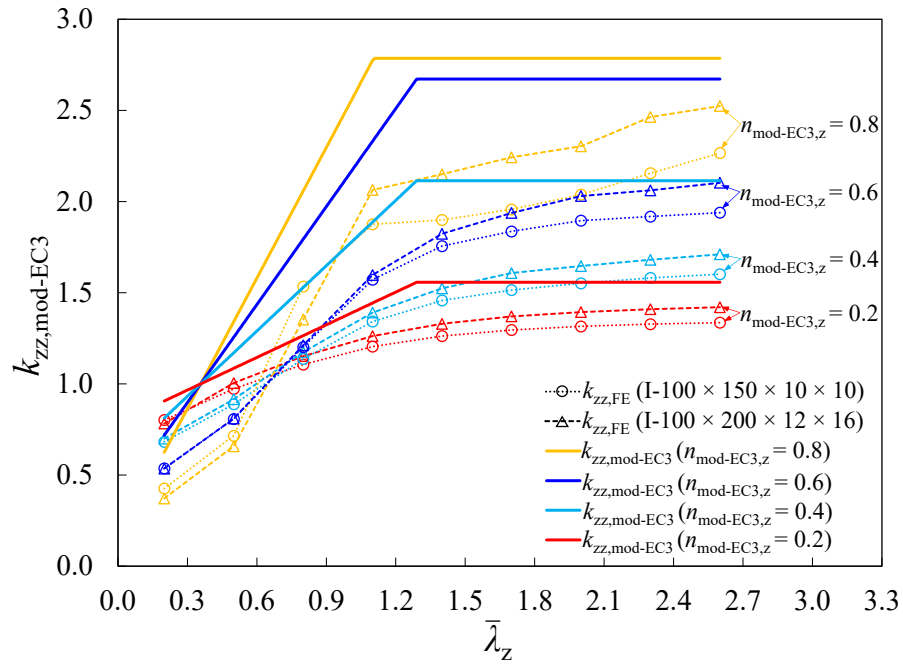


(b) Compression plus minor axis bending

Figure 5.10: Calibration of the modified interaction factors $k_{yy,mod-EC3}$ and $k_{zz,mod-EC3}$ for S355 steel welded I-section beam-columns.

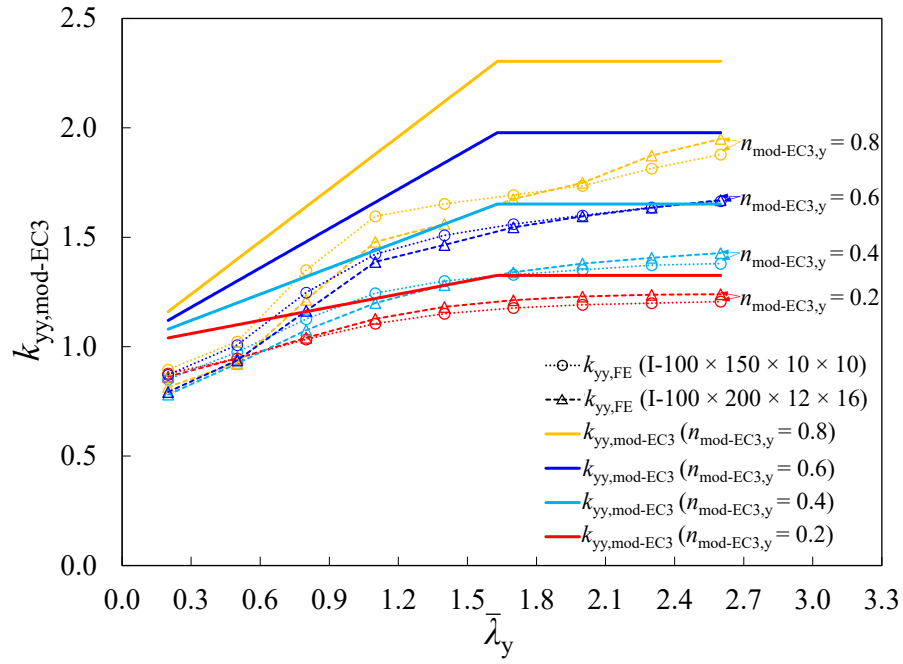


(a) Compression plus major axis bending

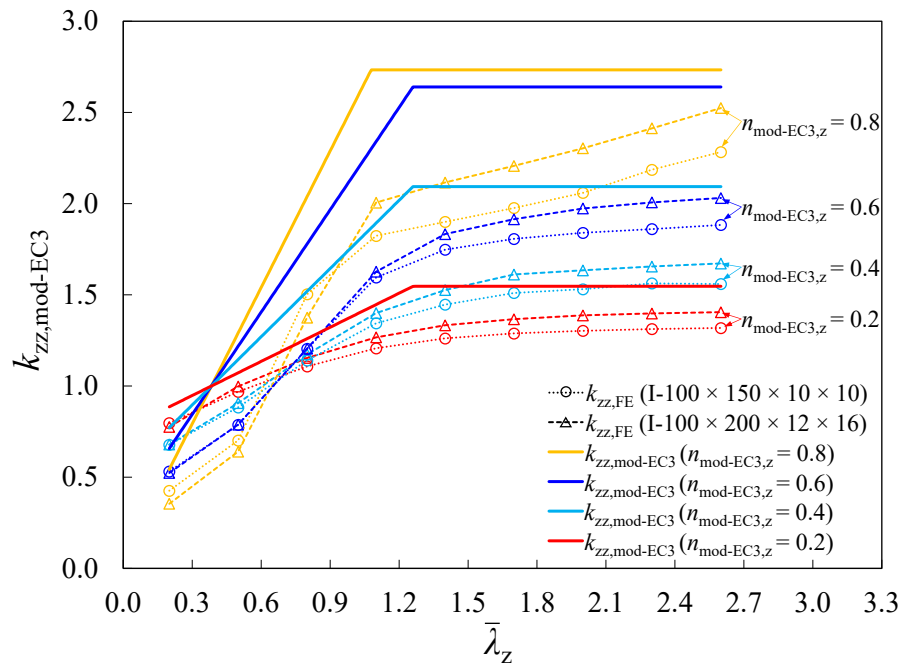


(b) Compression plus minor axis bending

Figure 5.11: Calibration of the modified interaction factors $k_{yy,\text{mod-EC3}}$ and $k_{zz,\text{mod-EC3}}$ for S460 steel welded I-section beam-columns.

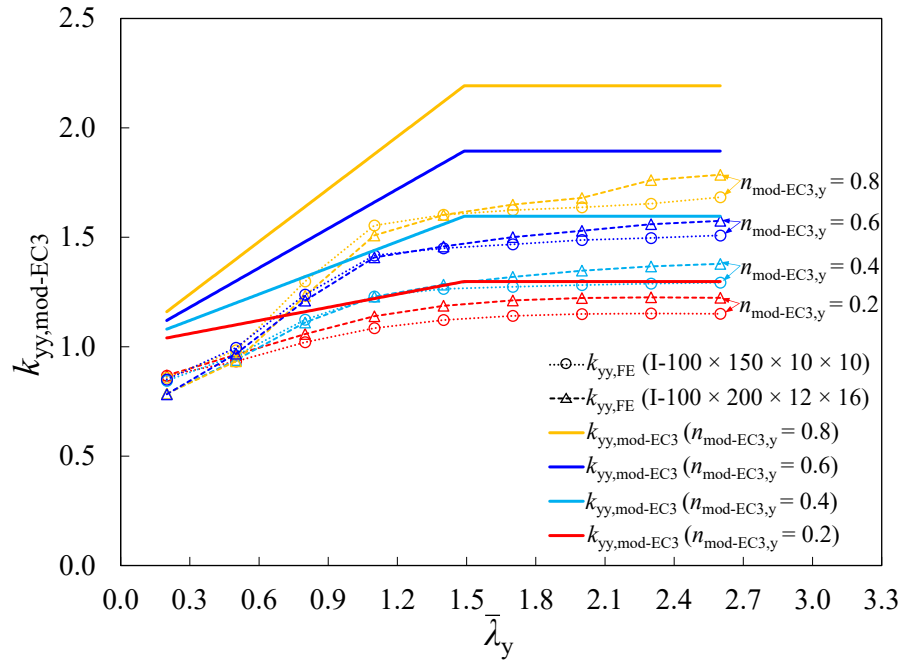


(a) Compression plus major axis bending

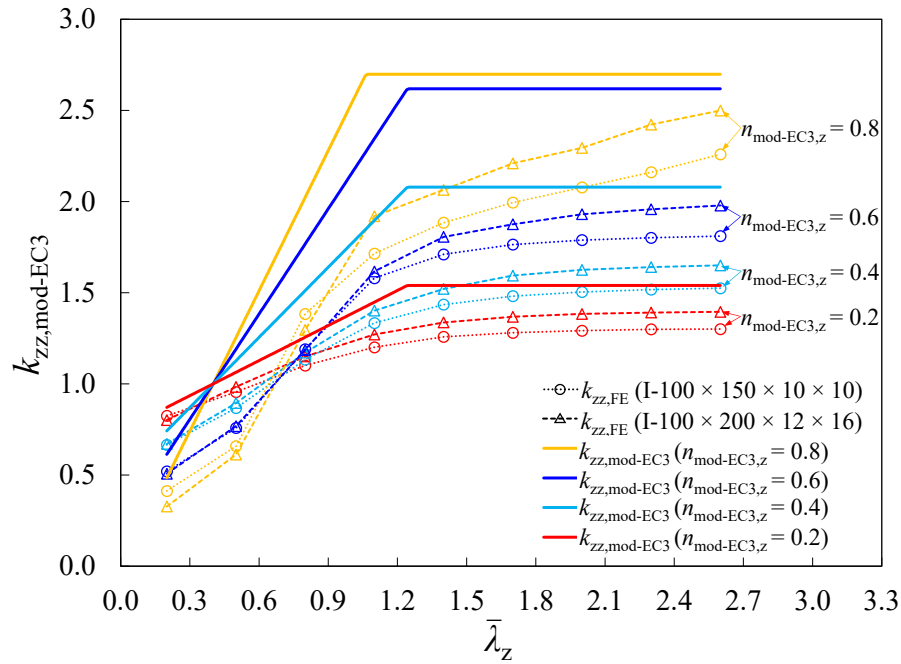


(b) Compression plus minor axis bending

Figure 5.12: Calibration of the modified interaction factors $k_{yy,mod-EC3}$ and $k_{zz,mod-EC3}$ for S690 steel welded I-section beam-columns.



(a) Compression plus major axis bending

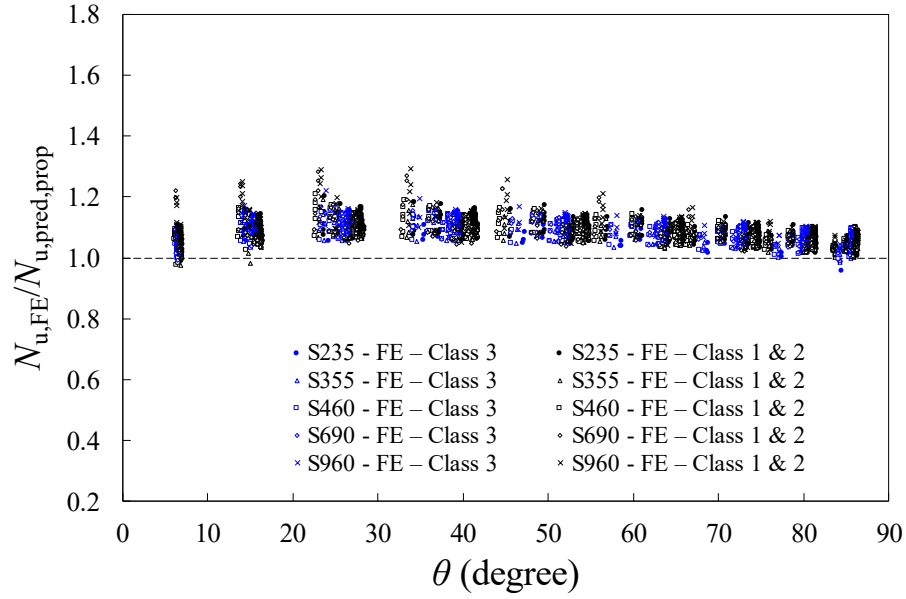


(b) Compression plus minor axis bending

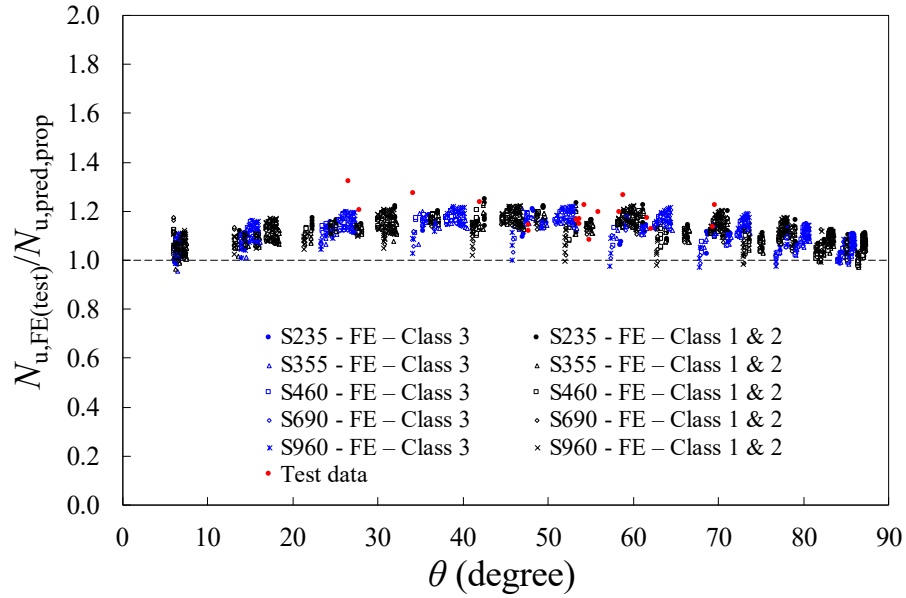
Figure 5.13: Calibration of the modified interaction factors $k_{yy,mod-EC3}$ and $k_{zz,mod-EC3}$ for S960 steel welded I-section beam-columns.

5.4.3 Assessment of new design proposals

The accuracy of the newly developed design proposals for NSS and HSS welded I-section beam-columns is evaluated through comparisons against the existing test data (Ma et al., 2017b; Yang et al., 2018) and numerical results generated from the parametric studies described in Section 5.2.3. The ratios of $N_{u,FE(test)}/N_{u,pred,prop}$ are plotted against the corresponding angle parameter θ in Figure 5.14(a) and (b) for beam-columns under compression plus major and minor axis bending, respectively. It can be observed from Figure 5.14(a) and (b) that the new design proposals result in more accurate and less scattered ultimate resistance predictions for NSS and HSS non-slender welded I-section beam-columns than the current EC3 and AISC design provisions. As indicated in Table 5.3 and Table 5.4, the mean ratios of $N_{u,FE(test)}/N_{u,pred,prop}$ are equal to 1.085 and 1.115 for beam-columns subjected to compression plus major and minor axis bending, respectively, with corresponding COV values equal to 0.035 and 0.048, which are significantly reduced compared to those of the current EC3 and AISC design provisions, highlighting the ability of the proposed method to provide more consistent resistance predictions for beam-columns made of steel grades varying from S235 to S960. Moreover, the new design proposals predict overall safe-sided ultimate resistance predictions for NSS and HSS welded I-section beam-columns, with only a very limited number of predictions on the unsafe side, as shown in Figure 5.14. The significant improvements of the new design proposals over the existing EC3 and AISC design provisions are attributed to the adoption of the more accurate compression and bending end points, as well as the more accurate interaction curves calibrated against the numerically obtained results.



(a) Compression plus major axis bending



(b) Compression plus minor axis bending

Figure 5.14: Comparisons of FE and test results with resistance predictions determined using the new design proposals for NSS and HSS non-slender welded I-section beam-columns under combined loading.

5.5 Reliability analysis

The reliability of the new design proposals for NSS and HSS non-slender welded I-section beam-columns is investigated through statistical analyses in line with EN 1990 (2002) using the numerically obtained data from the parametric studies as well as the test data collected from the literature (Ma et al., 2017b; Yang et al., 2018). The key variability parameters of the basic

material and geometric properties are taken from Annex E of prEN 1993-1-1 (2019), as listed in Table 5.5, where X_m is the mean value of the variable, X_n is the nominal value of the variable and V_x is the corresponding COV of the variable. The COV of the cross-sectional area V_A can be determined on the basis of the variability parameters of the basic cross-sectional dimensions H , B , t_f and t_w following the method outlined in (Afshan et al., 2015), which is, on average, equal to 0.02 for all the investigated welded I-sections.

Table 5.5: Variability parameters of material and geometric properties specified in prEN 1993-1-1 (2019)

Parameter	X_m/X_n	V_x
f_y (S235)	1.25	0.055
f_y (S355)	1.20	0.050
f_y (S460)	1.15	0.045
f_y (above S460)	1.10	0.035
E	1.00	0.030
H	1.00	0.009
B	1.00	0.009
t_f	0.98	0.025
t_w	1.00	0.025

The beam-column design formulae can be rewritten in the form of Equation (5.26), where C_0 is a constant coefficient, and C_1 , C_2 and C_3 are the coefficients indicating the level of participation of the material yield strength f_y , cross-sectional area A and Young's modulus E , respectively, in the beam-column resistance predictions; these coefficients can be calculated following the method detailed in Afshan et al. (2015).

$$N_{b,R} = C_0 f_y^{C_1} A^{C_2} E^{C_3} \quad (5.26)$$

The combined COV of the material and geometric basic variables V_{rt} can be then calculated using:

$$V_{rt} = \sqrt{(C_1 V_{f_y})^2 + (C_2 V_A)^2 + (C_3 V_E)^2} \quad (5.27)$$

Other key statistical parameters are also presented in Table 5.6 and Table 5.7 for beam-columns made of varying steel grades under combined compression plus major axis bending and minor axis bending, respectively, where n is the number of the numerical and experimental data points used in the reliability analysis, b is the correction factor, $k_{d,n}$ is the design fractile factor depending on the value of n , V_{δ} is the COV of the ratio of $N_{u,FE(test)}/N_{u,pred,prop}$ and γ_{M1}^* is the required partial safety factor. Note that the correction factor b is determined using Equation (5.28) instead of the least squares method recommended in EN 1990 (2002) in order to prevent b being biased towards the numerical results of beam-columns with higher ultimate resistances (Meng et al., 2020; Walport et al., 2020), where $r_{e,i}$ represents the numerical ultimate resistance and $r_{t,i}$ represents the theoretical ultimate resistance determined using the new design proposals presented in Section 5.4.

$$b = \frac{1}{n} \sum_{i=1}^n \frac{r_{e,i}}{r_{t,i}} \quad (5.28)$$

Table 5.6: Key reliability analysis results for the new design proposals for NSS and HSS welded I-section beam-columns under compression plus major axis bending.

Steel grade	No. of FE data	b	$k_{d,n}$	V_{δ}	γ_{M1}^*
S235	729	1.076	3.054	0.033	0.951
S355	729	1.077	3.054	0.032	0.970
S460	729	1.081	3.054	0.031	0.996
S690	729	1.090	3.054	0.035	1.018
S960	729	1.103	3.054	0.034	0.998
Overall	3645	1.085	3.042	0.034	1.014

Table 5.7: Key reliability analysis results for the new design proposals for NSS and HSS welded I-section beam-columns under compression plus minor axis bending.

Steel grade	No. of FE (test) data	b	$k_{d,n}$	V_{δ}	γ_{M1}^*
S235	729	1.119	3.054	0.045	0.944
S355	729	1.108	3.054	0.047	0.967
S460	729 (14)	1.113	3.054	0.047	1.059
S690	729 (4)	1.113	3.054	0.049	1.031
S960	729	1.120	3.054	0.051	1.026
Overall	3645 (18)	1.115	3.042	0.048	1.026

It can be seen from Table 5.6 and Table 5.7 that the required partial safety factors γ_{M1}^* for the newly proposed design method are lower than or very close to the target value of 1.0 (EN 1993-

1-1, 2019) for welded I-section beam-columns with varying steel grades. It can thus be concluded that the newly proposed design method can be safely applied to the design of NSS and HSS non-slender welded I-section beam-columns under compression plus uniaxial bending using the current EC3 partial safety factor of 1.0.

5.6 Conclusions

A comprehensive and systematic numerical modelling programme has been conducted in this chapter to explore the behaviour and design of NSS and HSS non-slender welded I-section beam-columns subjected to compression plus uniaxial bending. FE models that can closely replicate the structural response of NSS and HSS welded I-section beam-columns were established and validated against existing test results from the literature (Ma et al., 2017b; Yang et al., 2018). The validated FE models were then utilised to conduct parametric studies considering a wide range of cross-section geometries, member slendernesses, steel grades and loading combinations. The numerically obtained results, as well as the test data collected from the literature (Ma et al., 2017b; Yang et al., 2018), were employed to evaluate the accuracy of the current beam-column design methods provided in prEN 1993-1-1 (2019), EN 1993-1-12 (2007) and AISC 360-16 (2016). It was found that both the EC3 and AISC design methods yield somewhat scattered ultimate resistance predictions with inaccuracies arising in the endpoints of the interaction curves and the interaction curves themselves. To overcome the shortcomings in the existing design provisions, new beam-column design proposals were established, featuring the adoption of the newly proposed column buckling design method in Chapter 3 and the Continuous Strength Method (CSM) (Yun et al., 2018a) for the calculation of the compression and bending end point resistances, respectively, together with newly devised interaction curves anchored to these more accurate end points. The new design proposals were shown to provide more accurate, consistent and generally safe-sided resistance predictions than the current design methods for non-slender welded I-section beam-columns made of steel grades varying from S235 to S960. The reliability of the new design proposals was also assessed by means of statistical analysis following the procedures provided in EN 1990 (2002), confirming that the current EC3 partial safety factor of unity can be safely used.

Chapter 6

Tests on high strength steel frames

6.1 Introduction

A comprehensive experimental study comprising tests on a total of eight steel frames made of S690 welded I-sections has been conducted in the Structures Laboratory at Imperial College London and is presented herein. Advanced experimental techniques, including digital image correlation, were employed to gain a thorough insight into the response of the tested frames. Details of the test specimens, setup and procedures are fully described, while the results are reported and analysed. The main objective of the present study is to generate experimental data on HSS indeterminate frames, which can be used in future studies as benchmarks for the validation of advanced numerical models and design approaches.

6.2 Experimental program

6.2.1 Frame specimens

A total of eight two-dimensional, single bay, single storey, unbraced rectangular frames was tested. For each of the eight tested frames, the same welded I-section was used for both the columns and beam. The frame specimens were categorized into two series, with each series consisting of four identical frames, according to their cross-section (CS) profile: Series 1 (CS1 HSS-I-65×116×8×8) and Series 2 (CS2 HSS-I-80×136×8×8). Note that the cross-sections of the frame specimens were selected so that the failure load of each frame was within the limits

of the actuator capacities and the maximum deformation of each frame was less than the available stroke of the hydraulic actuators, as described in Section 6.3. The cross-section identifier is composed of four letters “HSS-I” designating an HSS welded I-section made of S690 steel, followed by the nominal dimensions of the I-section in millimetres, i.e. flange width $B \times$ outer section depth $H \times$ flange thickness $t_f \times$ web thickness t_w , as illustrated in Figure 6.1. The two different welded I-sections were fabricated from 8 mm-thick quenched and tempered S690 steel plates by means of gas metal arc welding with a nominal weld leg length (t_{weld}) of 5.6 mm (i.e. a weld throat thickness of 4 mm), as shown in Figure 6.1. Note that the two investigated welded I-sections are both Class 1 under pure compression according to the slenderness limits specified in EN 1993-1-12 (2007) but have different local flange and web slenderness ratios to enable the influence of local slenderness on the key issues of rotation capacity and potential for plastic redistribution to be studied. The key material properties and full-range stress-strain responses of the studied S690 steel have been reported in Table 3.4 and shown in Figure 3.5.

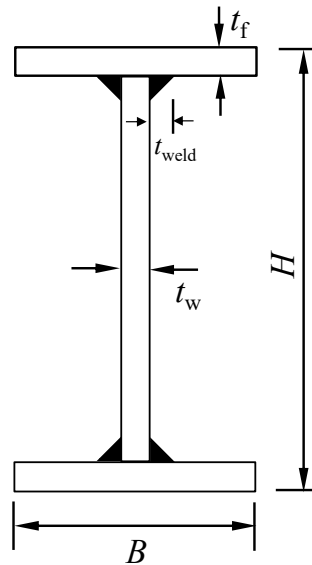


Figure 6.1: Notation of welded I-sections.

Figure 6.2 shows the configuration of the test frame specimens; the span of the frames (i.e. the distance between the column centrelines) was 3.168 m and the height of the frames (i.e. from the base of the columns to the centreline of the beam) was 2 m. The average measured geometric dimensions of the cross-section of each frame specimen are presented in Table 6.1;

the cross-sectional properties of each frame specimen, calculated based on the average measured values, are also presented in Table 6.1, where A is the cross-sectional area, W_{el} and W_{pl} are the elastic and plastic section moduli, respectively, and M_{el} and M_{pl} are the elastic and plastic bending moment capacity, respectively. The frame specimens are labelled according to their cross-section profile and loading condition, as shown in Table 6.1, in which ‘V’ indicates that the frame is subjected to pure vertical load, ‘H’ indicates that the frame is subjected to pure horizontal load, and ‘V&H-1’ and ‘V&H-2’ indicate that the frame is subjected to combined vertical and horizontal loads, with different vertical-to-horizontal load ratios. Note that in the present study, unless otherwise stated, the cross-sectional properties are given with respect to the major axis.

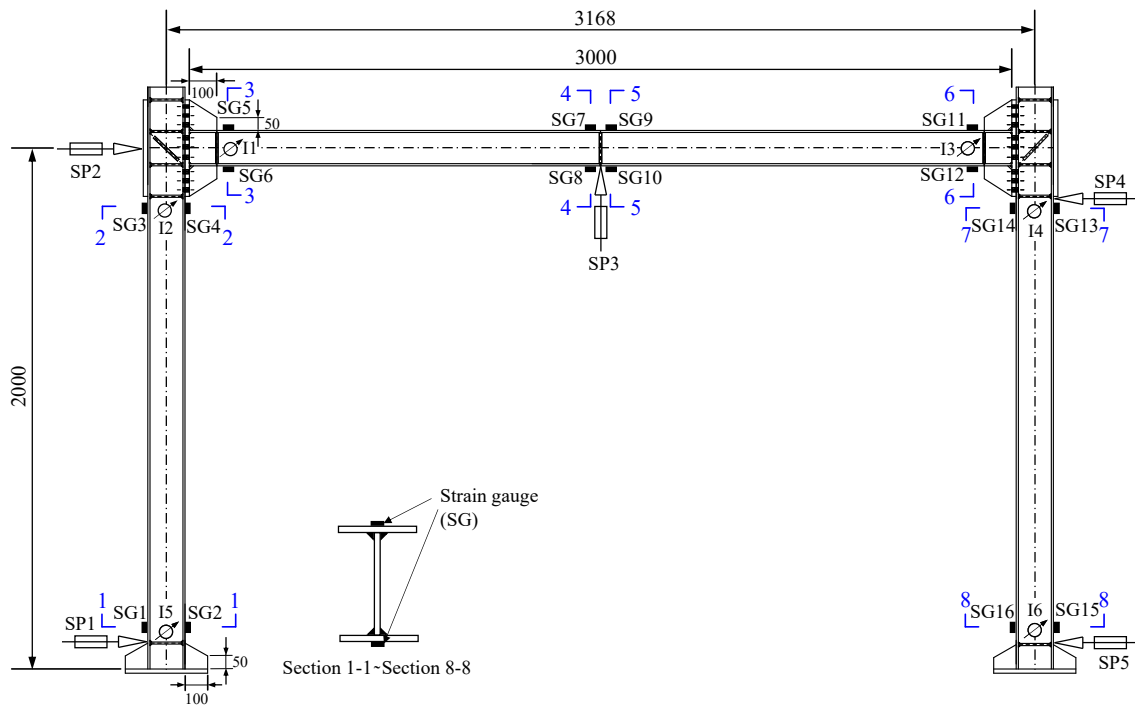
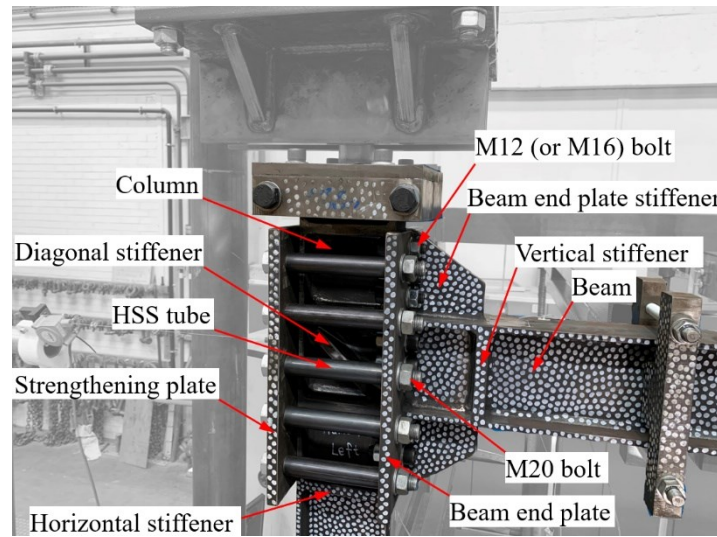


Figure 6.2: Configuration of frame specimens and instrumentation, including strain gauges (SG), string potentiometers (SP) and inclinometers (I). Dimensions are shown in mm.

Table 6.1: Average measured geometric dimensions and cross-sectional properties of frame specimens.

Series	Frame label	B mm	H mm	t_f mm	t_w mm	t_{weld} mm	A mm ²	W_{el} mm ³	W_{pl} mm ³	M_{el} kNm	M_{pl} kNm
Series 1	HSS-I-65×116-V	63.64	115.20	8.31	8.21	5.21	1921.3	6.607×10^4	7.905×10^4	51.70	61.86
	HSS-I-65×116-H	64.10	115.73	8.41	8.29	5.52	1959.1	6.770×10^4	8.103×10^4	52.98	63.41
	HSS-I-65×116-V&H-1	64.08	115.79	8.36	8.22	4.96	1934.9	6.696×10^4	8.007×10^4	52.40	62.66
	HSS-I-65×116-V&H-2	64.00	115.30	8.33	8.24	4.87	1926.5	6.632×10^4	7.933×10^4	51.90	62.08
Series 2	HSS-I-80×136-V	79.06	136.77	8.47	8.39	5.61	2407.6	1.014×10^5	1.197×10^5	79.35	93.66
	HSS-I-80×136-H	79.63	136.46	8.51	8.42	5.56	2422.8	1.020×10^5	1.203×10^5	79.80	94.15
	HSS-I-80×136-V&H-1	78.20	136.76	8.36	8.26	5.11	2351.2	9.893×10^4	1.167×10^5	77.42	91.35
	HSS-I-80×136-V&H-2	79.02	136.60	8.35	8.28	4.92	2360.8	9.939×10^4	1.172×10^5	77.77	91.72

**Figure 6.3:** Beam-to-column connection.

All connections employed in the present test programme, including the extended end plate beam-to-column connections and the base connections (as detailed in Section 6.3.2), were designed to be moment-resisting and near rigid, as shown in Figure 6.3. Note that in Figure 6.3, the white paint on the surface of the beam-to-column connection are a speckle pattern for the digital image correlation (DIC) technique that was utilized during the frame testing to measure the full field displacement of the specimens. The beam-to-column connections comprised a 16-mm thick beam end plate, made of S690 steel, which was fillet welded to the beam end and bolted to the column flange using eighteen preloaded high strength M12 or M16 bolts for the frame specimens composed of the smaller (I-65×116×8×8) or larger (I-80×136×8×8) welded I-section, respectively. The beam end plate has a greater width than that of its connected column,

thereby enabling the beam-to-column connection to be further strengthened by means of bolting the beam end plate to a strengthening plate attached to the outer side of the column flange using ten preloaded high strength M20 bolts, as illustrated in Figure 6.3. Grade 8.8 bolts were used in the connections of the first tested frame specimen (i.e. HSS-I-100-65-V), while higher grade (12.9) bolts were adopted in the remaining frame test specimens. All bolts were tightened to predetermined torque values according to the requirements set out in EN 1090-2 (2018). Tightening torques of 425 Nm, 210 Nm, 88 Nm were applied to the Grade 8.8 M20, M16 and M12 bolts respectively, and 720 Nm, 350 Nm, 150 Nm to the Grade 12.9 M20, M16 and M12 bolts, respectively. Each of the M20 bolts was housed within a steel tube, positioned between the beam end plate and the stiffening plate (see Figure 6.3), to prevent the inward bending of the two plates due to the preload in the M20 bolts. The dimensions and the layout of the bolts for the beam-to-column connections of the frame specimens composed of the smaller (I-65×116×8×8) and larger (I-80×136×8×8) welded I-section are shown in Figure 6.4(a) and (b), respectively. A series of stiffeners (i.e. horizontal stiffeners, vertical stiffeners, diagonal stiffeners and beam end plate stiffeners, as indicated in Figure 6.3 and Figure 6.4) were cut from 16 mm thick S690 steel plates and welded onto the frame panel zones and beam end plates to increase the strength and stiffness of the connections. Note that the presence of the beam end plate stiffeners also had the effect of shifting the plastic hinge locations away from the beam ends to the beam sections where the stiffeners terminated; the column end plate stiffeners had the same effect, as described in the next section.

An additional 10 mm-thick bearing plate was inserted between the beam end plate and the column flange for the frame specimens composed of the smaller welded I-section (I-65×116×8×8), as shown in Figure 6.4(a), to maintain the same distance between the centrelines of the two columns as for the frame specimens with the larger welded I-section (I-80×136×8×8); this also aided the installation process.



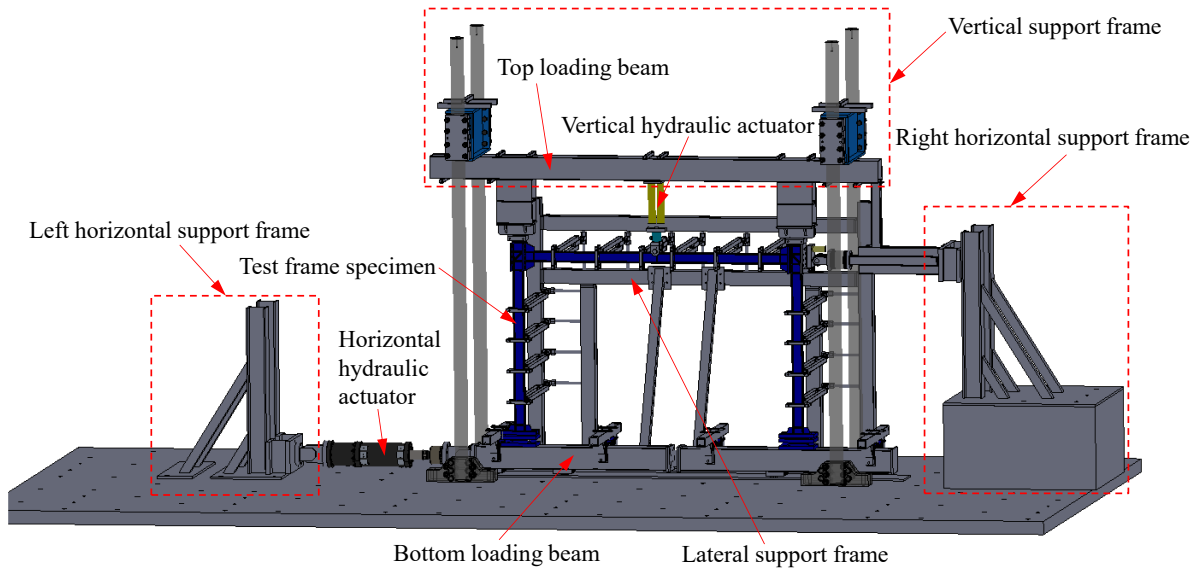


Figure 6.5: Schematic solidworks model of frame test setup.

6.3.1 Loading conditions

Different loading conditions were applied to the four frame specimens within each of the two test series, with one subjected to vertical load V only applied at the mid-span of the beam, one subjected to horizontal load H only applied to the bottom loading beam and the other two subjected to different combinations of the two (V and H). For the frames subjected to combined loading, the vertical load V_{test} was first applied at the mid-span of the beam until the pre-determined load was reached (i.e. 100 kN and 130 kN for HSS-I-65×116-V&H-1 and HSS-I-65×116-V&H-2 respectively and 150 kN and 210 kN for HSS-I-80×136-V&H-1 and HSS-I-80×136-V&H-2 respectively). The vertical load V_{test} was then held constant while the horizontal load H_{test} was applied at the column base through the bottom loading beam until either collapse of the frame or excessive deformation occurred. For all the frame tests, the vertical load was applied under load control at a slow rate of approximately 5 kN/min, while the horizontal load was introduced under displacement control at approximately 2 mm/min.

The vertical load was applied using two 232 kN (24 tonnes) hydraulic actuators, which were mounted on the bottom flange of the top loading beam, in alignment with the mid-span of the beam of the frame specimen. The top loading beam was a 250 × 250 × 8 mm cold-formed S700 steel square hollow section (SHS), which was fixed to the vertical support frame, as shown in

Figure 6.5. The two vertical hydraulic actuators, placed next to each other in parallel, were fed by a single pump in order to ensure even loading. The vertical load was measured using a calibrated load cell positioned under the vertical hydraulic actuators and applied through a steel roller, as shown in Figure 6.6. A pair of stiffeners was welded to the frame specimen at the vertical loading point to prevent web crippling due to the concentrated vertical load, as also shown in Figure 6.6.

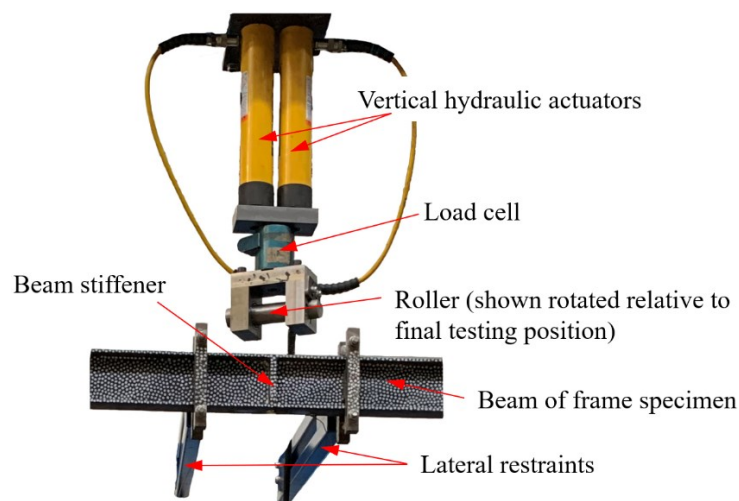


Figure 6.6: Vertical loading system.

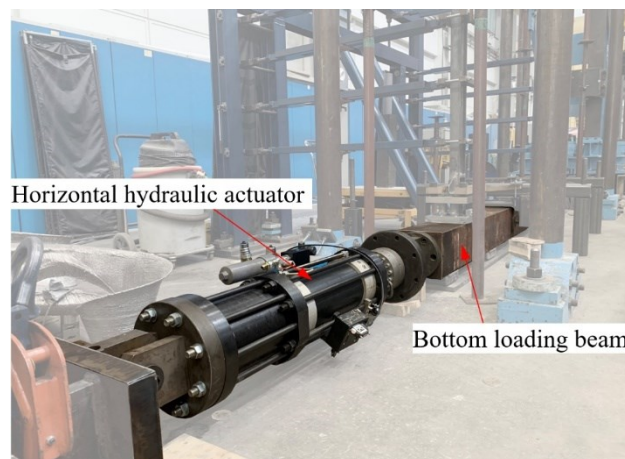
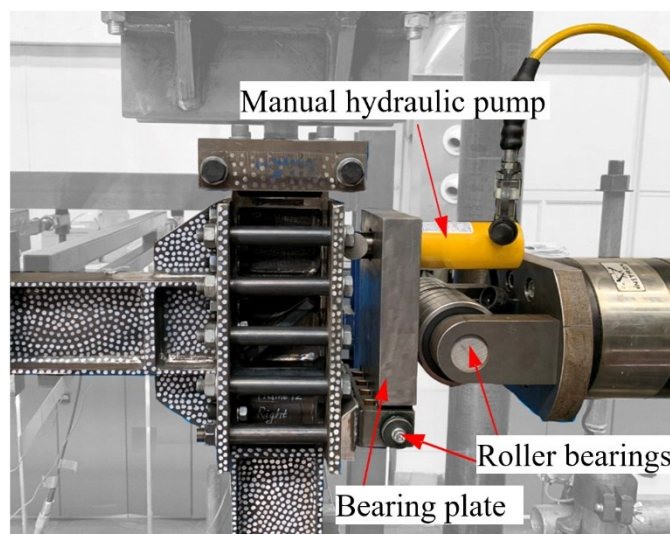


Figure 6.7: Horizontal loading system.

The horizontal load was applied to the bottom loading beam by a 250 kN hydraulic actuator, as shown in Figure 6.7, which was mounted on the left horizontal support frame, as illustrated

in Figure 6.5. The frame specimens were fixed to the bottom loading beam, which was seated on several greased roller bearings, allowing the frames to move in-plane with minimal friction. The frame specimens were horizontally restrained at the top of the right-hand column (see Figure 6.5 and Figure 6.8) by a horizontal support. The horizontal support included a manual hydraulic pump, a bearing plate and a set of steel rollers, as illustrated in Figure 6.8. The manual hydraulic pump was fixed to the bearing plate, and the pump head (featuring the roller bearings) was attached to the surface of the strengthening plate prior to testing, as shown in Figure 6.8(a). For the frame specimens that were loaded horizontally, the manual hydraulic pump was used to adjust the position of the bearing plate such that it remained vertical during testing, as shown in Figure 6.8(b); this ensured that the reaction remained horizontal, despite the rotation of the right-hand beam-to-column connection. The load provided by the manual hydraulic pump was increased as the horizontal load increased, approximately to half of the applied horizontal load. The horizontal support provided a “hinge-like” boundary condition at the top of the right-hand column. The horizontal load was applied at the base level, allowing the vertical actuator to remain stationary during testing, rather than requiring it to move laterally with the frame. This loading configuration is statically equivalent to applying the horizontal load at the right-hand beam-to-column connection and restraining lateral movements at the column bases. Note that the bottom loading beam was of the same cross-section profile as the top loading beam, but was filled with concrete to prevent any local failure of the tube due to the concentrated loading.



(a)

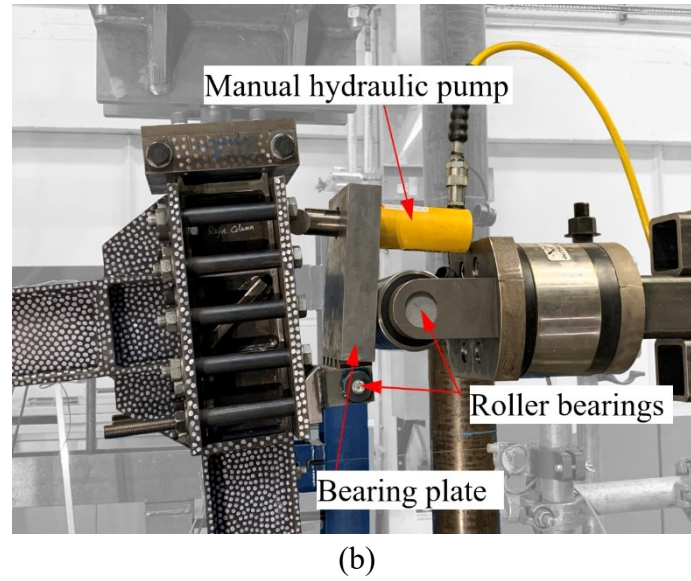


Figure 6.8: Horizontal support (a) prior to testing; and (b) during testing.

6.3.2 Load cells

Three specially fabricated load cells, similar to those used by Young and Rasmussen (2003) and Arrayago et al. (2020a), were employed to measure the reaction forces and moments at the column bases. Each of the load cells consisted of two 38 mm-thick steel plates and four cylindrical steel studs (80 mm in depth and 50 mm in diameter) welded in between the two plates, as illustrated in Figure 6.9. Each of the four steel studs was equipped with four strain gauges, as shown in Figure 6.9, to measure the longitudinal strains at different locations; these strains were used to determine the applied load and bending moment.

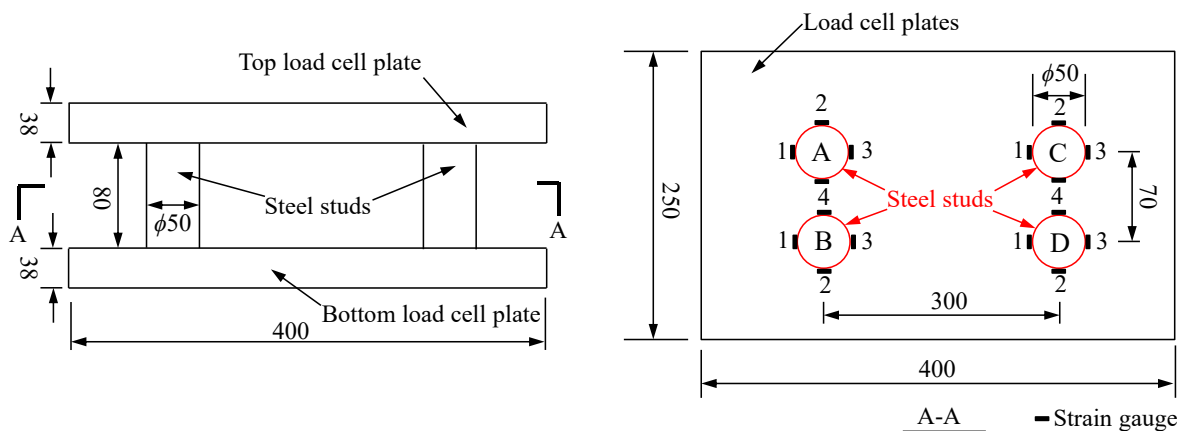


Figure 6.9: Dimensions of load cells in mm and arrangement of strain gauges.

The load cells were calibrated prior to fixing them to the frames. Two load cells were used to measure the vertical reaction loads and bending moments at the column bases as shown in Figure 6.10(a), while a third load cell was welded into the middle of the bottom loading beam, as shown in Figure 6.10(b), to determine the horizontal reaction loads (i.e. shear forces) at the column bases. As illustrated in Figure 6.11, the end plate of each column was bolted to the fabricated load cell using four Grade 8.8 M16 bolts. The bottom load cell plate at each column base was welded to the bottom loading beam, as shown in Figure 6.10(a). To enhance the rigidity of the column base connections, a 38 mm-thick U-shaped strengthening plate made of S355 steel was placed on top of each column end plate and bolted to the top load cell plate by means of four Grade 8.8 M30 bolts, and stiffeners were also welded at each column base to strengthen the column end plates, as shown in Figure 6.11.

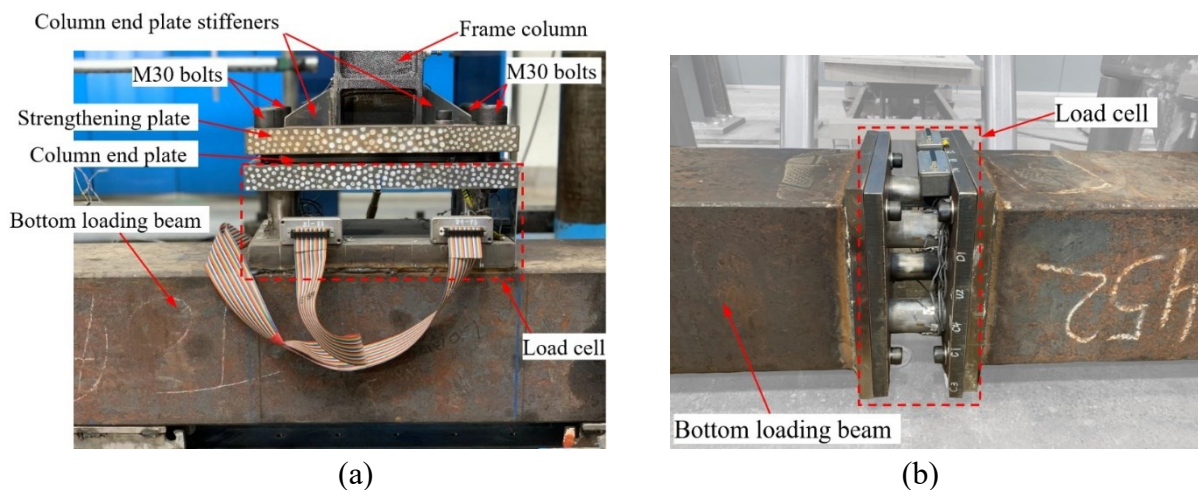


Figure 6.10: Load cells (a) beneath each column and (b) at middle of bottom loading beam.

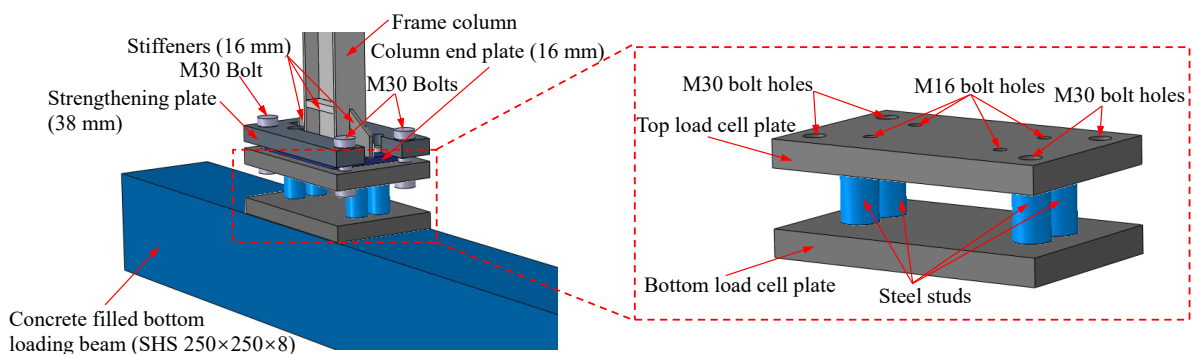
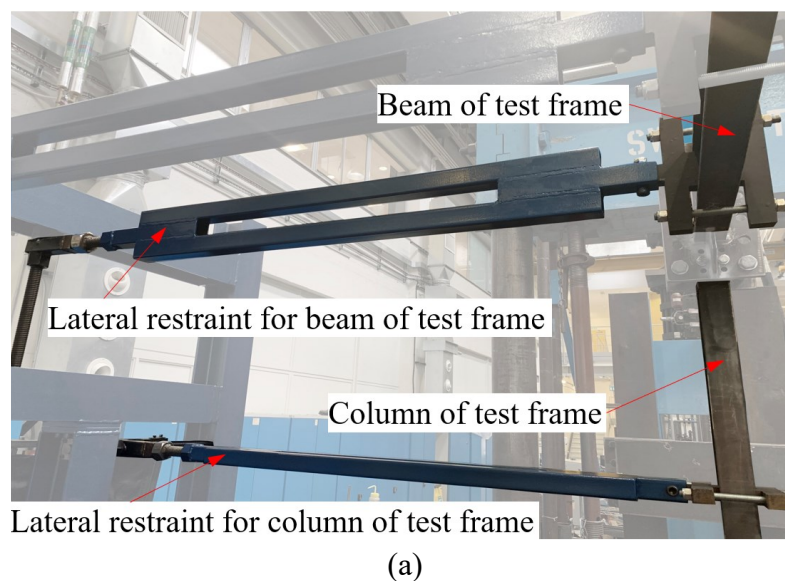


Figure 6.11: Schematic solidworks model of column base connection.

6.3.3 Lateral restraints

A bespoke restraint system was designed to prevent out-of-plane instability of the test frames without inducing any undesirable in-plane restraint. The restraint system comprised a total of 14 out-of-plane lateral restraints, among which 6 were used to provide lateral restraint to the beam of the test frames and 4 provided lateral restraint to each column, as shown in Figure 6.5. Each out-of-plane lateral restraint comprised two tee-shaped plates which were placed on either side of the web of the beam or column section of the frame specimens and connected using two M30 bolts, and a bracing member comprising two parallel S460 SHS 50×50×6.3 arms, as shown in Figure 6.12. The assembled tee-shaped plates were connected to the bracing member using an M24 bolt, which was greased to facilitate free rotation. Two 35 mm diameter Macalloy bars, as shown in Figure 6.12, were also employed in each lateral restraint; one allowed adjustment of the length of the lateral restraint as well as the rotation of the lateral restraint about the axis of the bar, while the other permitted the horizontal (for column lateral restraints) or vertical (for beam lateral restraints) movement of the lateral restraint as the frame specimen deformed during testing. A ratchet spanner was used to rotate the threaded bar manually, thereby enabling the horizontal (or vertical) movement of the lateral restraints to match that of the frames during testing, as shown in Figure 6.13.



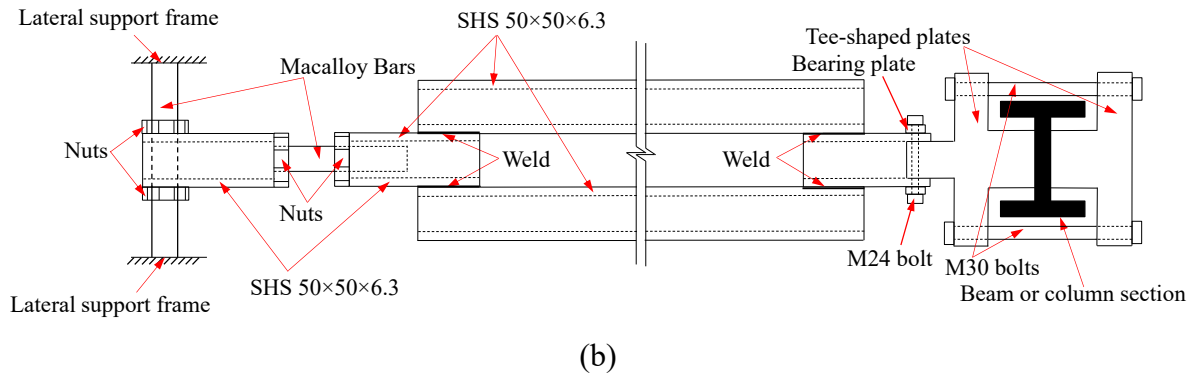


Figure 6.12: Out-of-plane lateral restraints for test frames (a) photo; and (b) schematic drawing.

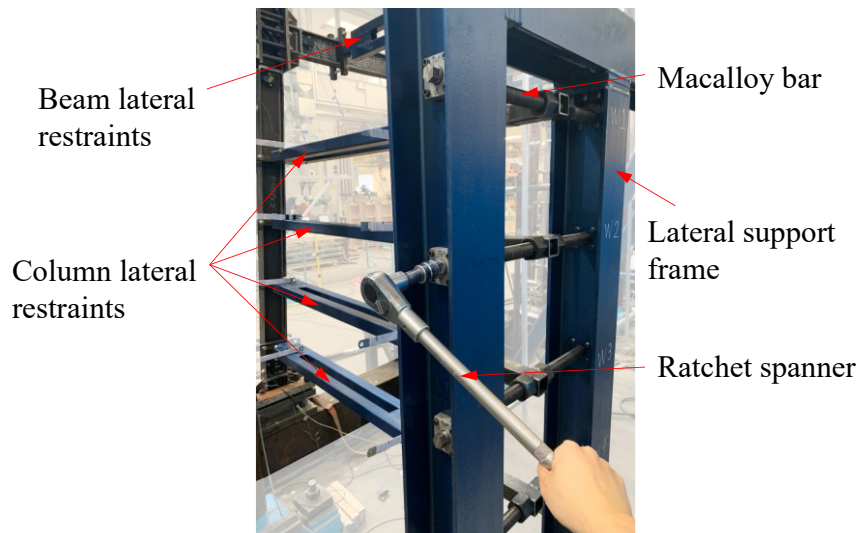


Figure 6.13: Adjustment of horizontal position of column lateral restraints.

A spirit level was placed on each beam lateral restraint to facilitate adjustment to ensure the restraint remained horizontal throughout testing. Similarly, the column lateral restraints were moved along with the displacements of the columns of the frame specimens at the corresponding heights. Figure 6.14 illustrates the methodology for ensuring the alignment of the column lateral restraint and the corresponding position on the column of the frame specimen. A pair of matching color-coded ‘rainbow’ rulers with lines at 2 mm intervals were mounted to both ends of each column lateral restraint, and a laser level was employed to generate the vertical line on the rulers. The restraints were adjusted such that the matching colours were aligned, ensuring that that restraints remained perpendicular to the plane of the frames. The adopted restraint system and the continuous manual adjustment approach

throughout testing ensured that lateral restraint was provided to the frames, while minimizing any influence on the in-plane response.

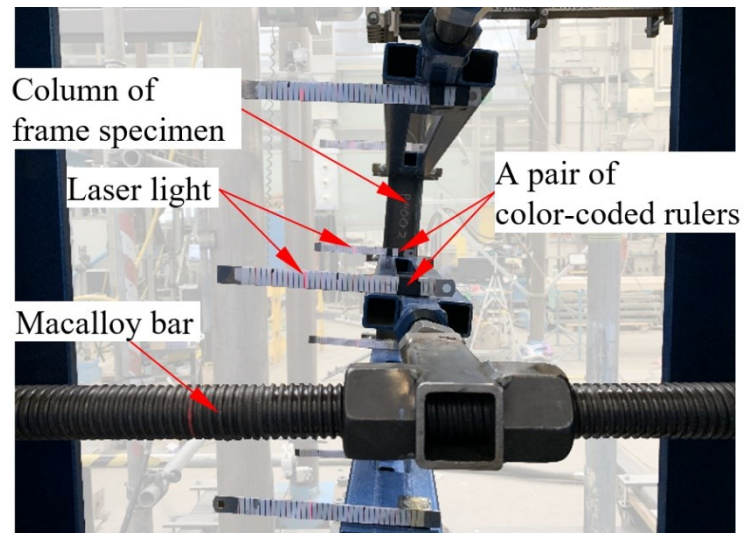


Figure 6.14: Use of laser to adjust the position of column lateral restraints.

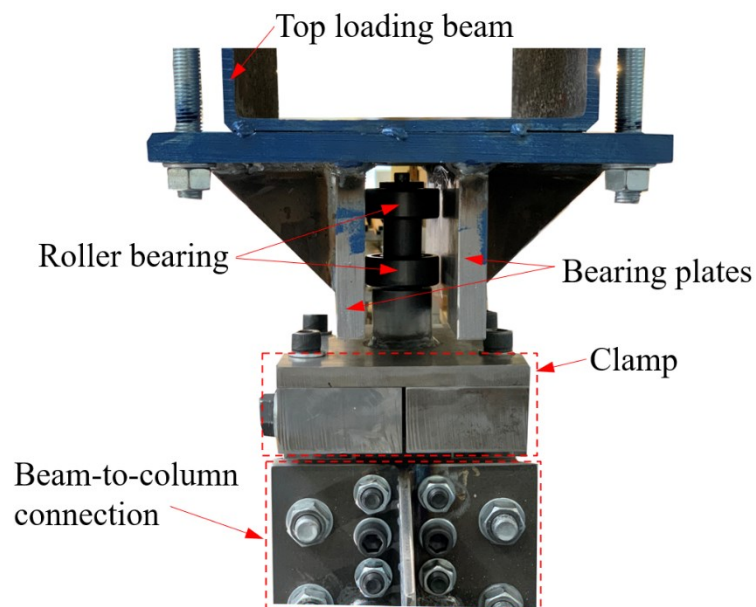


Figure 6.15: Lateral restraint at the top ends of the columns.

Out-of-plane lateral restraint was also provided at the top end of each column through a pair of bearing plates fastened to the top loading beam, as shown in Figure 6.15. A specially devised

roller bearing was fixed to the top of each column, the displacement of which was constrained to remain in-plane by the bearing plates. Specifically, the roller bearing was welded to a fabricated clamp, placed on the top of each column, which was composed of three steel plates and had an I-shaped groove at the bottom to accommodate the top end section of the column. The roller bearing was used in order to allow the frame specimen to deform in-plane with minimum, if any, friction being generated.

6.3.4 Instrumentation

Electrical resistance strain gauges were mounted on the top and bottom flanges at selected locations on the frame specimens to estimate bending moments and monitor the strain development histories during testing. As indicated in Figure 6.2, a total of eight beam and column sections (Section 1-1 to Section 8-8 in Figure 6.2) were monitored, located adjacent to (i.e. at a distance of 50 mm from the corresponding stiffened section) the base of each frame column, the beam-to-column connections and the mid-span of the beam, covering the most highly stressed regions of the frames under the different loading scenarios. String potentiometers (SP) were used to measure the in-plane vertical deflections at the mid-span of the beam (see SP3 in Figure 6.2) and the in-plane horizontal deflections at the base and top of each column (see SP1, SP2, SP4 and SP5 in Figure 6.2). Note that SP4 was placed below the strengthening plate of the right-hand beam-to-column connection for the frame specimens subjected to horizontal loading to avoid conflict with the horizontal restraint system. In addition, a total of six inclinometers (I) was employed on each frame, positioned close to the beam-to-column connections and the column base connections, as illustrated in Figure 6.2, to measure the rotation of the connections and to evaluate their rotational stiffnesses. The test outputs, including the applied loads and displacements from the hydraulic actuators, as well as the readings from the strain gauges, string potentiometers and inclinometers, were continuously monitored and recorded at 1-s intervals using the data acquisition system DATASCAN.

6.3.5 Digital Image Correlation (DIC)

Digital Image Correlation (DIC) was also employed to provide more comprehensive displacement and strain measurements. Compared with conventional instrumentation (e.g. strain gauges, transducers and inclinometers) which is commonly used to monitor deformations

at key discrete locations, DIC enables full-field measurements to be captured, making it ideal for use in structural testing. A four-camera LaVision DIC system was used for the image acquisition, as shown in Figure 6.16, among which two cameras with 8 mm focal length lenses (Cameras 1 and 2 in Figure 6.16) were used to monitor the deformation fields of the full frame specimens, while the other two cameras (Cameras 3 and 4 in Figure 6.16), which were equipped with longer focal length lenses of 35 mm to ensure sufficient resolution, were employed to record the surface strain field at the most highly stressed region (i.e. the region where the first plastic hinge formed), which could be either at the base of one column or at the mid-span of the beam depending on the loading scenario, throughout testing. Prior to testing, the front surface of all frame specimens was sprayed with a thin layer of black paint, on top of which different random white speckle patterns were drawn in order to achieve an average speckle size of 3-5 pixels in the acquired images from the two pairs of cameras. Consequently, a fine speckle pattern was applied to the most highly stressed region of each frame while a relatively coarse speckle pattern was used elsewhere, as illustrated in Figure 6.16. The output signals from the horizontal and vertical hydraulic actuators were fed into the DIC system through an analogue-to-digital converter, while images were captured at 2-s intervals and processed using the DaVis version 8.4.0 imaging software (LaVision 2017 (2017)). The measurements from the strain gauges, string potentiometers and inclinometers, as described in the previous subsection, were used to corroborate the DIC measurements at key locations, as discussed in the next section.

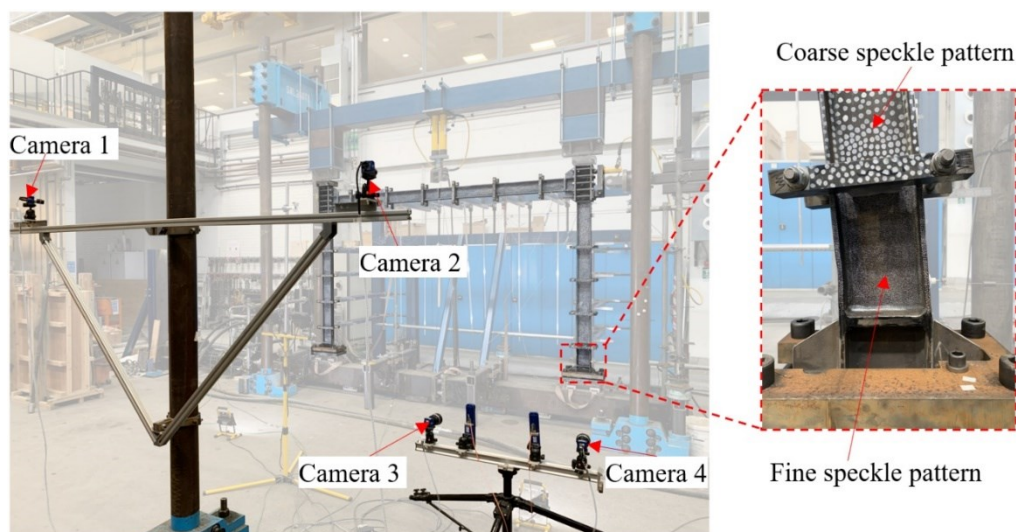
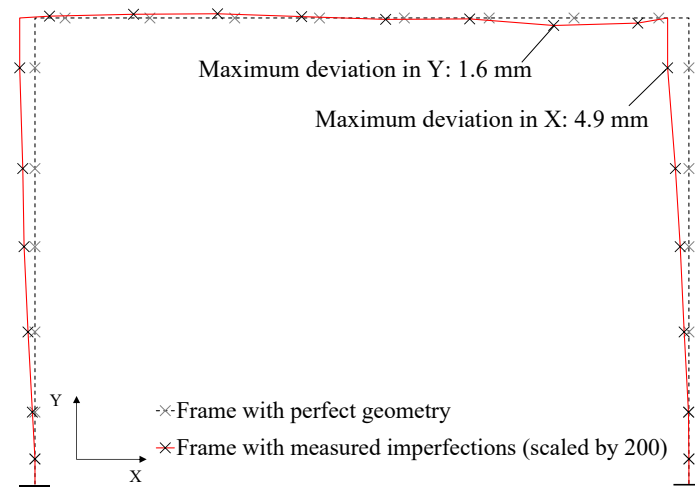


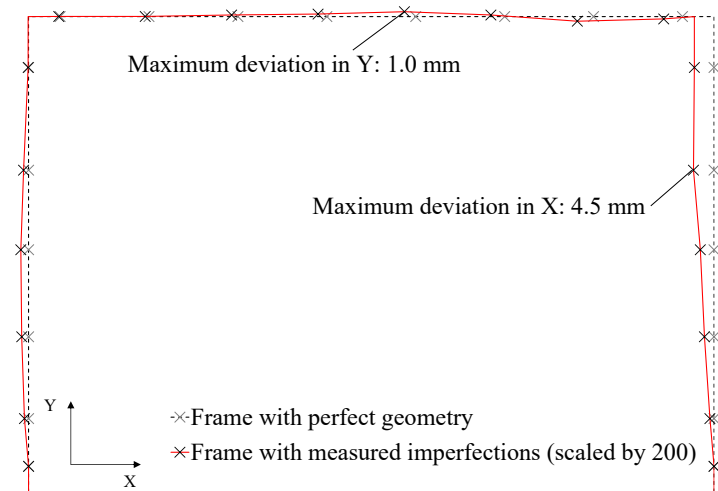
Figure 6.16: DIC setup and speckle patterns for frame tests.

6.3.6 Imperfection measurements

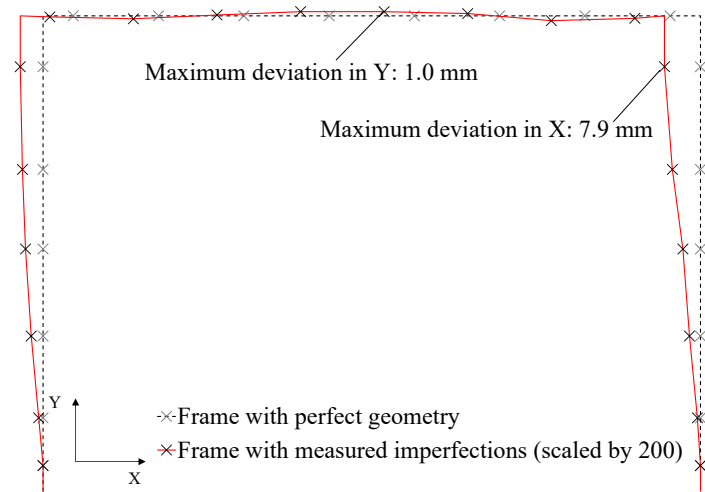
The initial imperfections of the assembled frame specimens, including out-of-plumbness of the frames, as well as out-of-straightness of the members, were measured. Plumb and horizontal lines were generated using a laser adjacent to the columns and beams of the frame specimens, respectively, to serve as reference lines from which deviations could be measured at a series of different locations, marked with cross-shaped points in Figure 6.17. The resulting initial in-plane imperfection measurements for all the frames are shown in Figure 6.17.



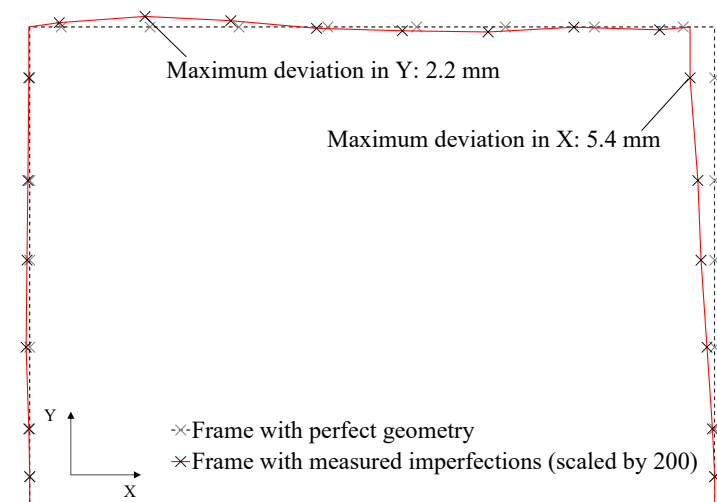
(a)



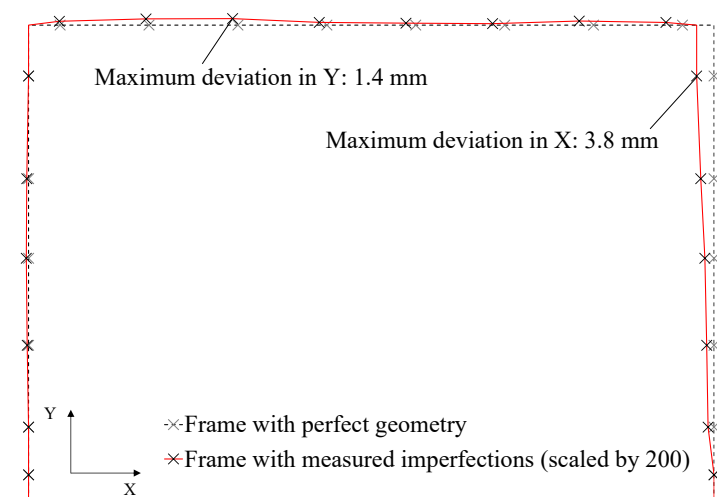
(b)



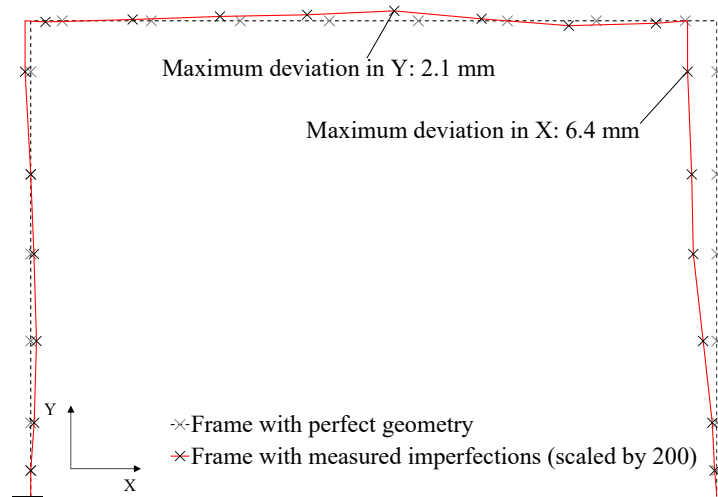
(c)



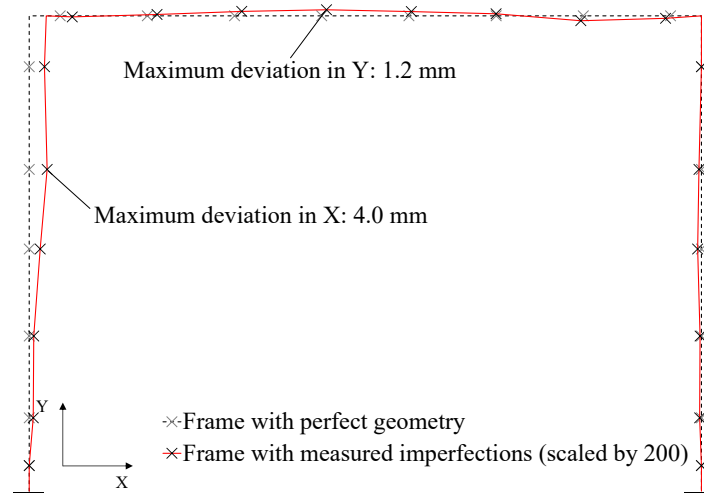
(d)



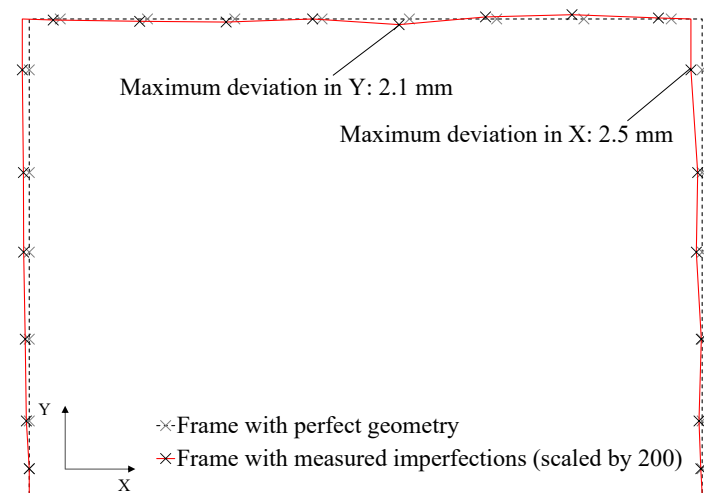
(e)



(f)



(g)



(h)

Figure 6.17: Measured in-plane imperfections for frame specimens (a) HSS-I-65×116-V; (b) HSS-I-65×116-H; (c) HSS-I-65×116-V&H-1; (d) HSS-I-65×116-V&H-2; (e) HSS-I-80×136-V; (f) HSS-I-80×136-H; (g) HSS-I-80×136-V&H-1; and (h) HSS-I-80×136-V&H-2.

According to EN 1090-2:2018 (2018), the mean inclination of all the columns in the same single storey of a building should be less than a prescribed erection tolerance of $h/500$, where h is the height of the column in metres. For the studied frames, this corresponds to the sum of the sway imperfection amplitudes of the left-hand and right-hand columns being less than 8 mm; this was achieved for all frames except for frame specimen HSS-I-65×116-V&H-1. The initial out-of-plane imperfections were also measured using the same method as described above, while the lateral positions of the frame specimens were adjusted using the lateral restraints (through the Macalloy bars positioned within the length of the lateral restraints, as shown in Figure 6.12) to ensure that the amplitudes of the out-of-plane imperfections were less than 2 mm for all frame specimens. These small amplitudes of out-of-plane imperfections, coupled with the fact that the frame specimens were laterally restrained throughout the testing, meant that the out-of-plane imperfections and deformations were considered to have negligible influence on the structural performance of the tested frame specimens.

6.4 Test observations and discussion

The key results measured from the frame experiments, including the ultimate collapse loads, failure modes, load-deformation histories and rotational stiffnesses of the different connections, are presented and discussed in this section.

6.4.1 Ultimate collapse loads

Table 6.2 summarizes the maximum vertical loads (or the pre-determined vertical loads for the frame specimens subjected to combined loading) $V_{\text{test,max}}$ and the corresponding vertical displacements at the mid-span of the beam of the frame specimens δ_{mid} , as well as the maximum horizontal loads $H_{\text{test,max}}$ and the corresponding horizontal displacements at the column bases δ_{base} . Note that the frame specimens subjected to horizontal load only (i.e. HSS-I-65×116-H and HSS-I-80×136-H) were terminated before reaching their ultimate loads due to the limited stroke of the horizontal hydraulic actuator of 250 mm; however, these maximum recorded loads ($H_{\text{test,max}}$) were deemed to be close to their corresponding ultimate loads since the tangent stiffnesses of the load-displacement curves at the corresponding maximum recorded load were approaching zero (dos Santos et al., 2018), as shown in Subsection 6.4.3.2.

Table 6.2: Summary of maximum applied loads and the corresponding displacements for frame specimens.

Series	Frame label	Vertical loading step		Horizontal loading step	
		$V_{\text{test,max}}$ kN	δ_{mid} mm	$H_{\text{test,max}}$ kN	δ_{base} mm
Series 1	HSS-I-65×116-V	161.9	173.5	-	-
	HSS-I-65×116-H	-	-	154.5	247.8
	HSS-I-65×116-V&H-1	101.3	38.7	128.2	207.6
	HSS-I-65×116-V&H-2	130.3	54.0	94.6	204.9
Series 2	HSS-I-80×136-V	260.6	143.4	-	-
	HSS-I-80×136-H	-	-	236.6	244.0
	HSS-I-80×136-V&H-1	152.7	37.0	173.9	174.6
	HSS-I-80×136-V&H-2	210.6	58.3	125.2	129.5

6.4.2 Failure modes

6.4.2.1 Frame specimens subjected to vertical load only

The failure modes of the tested frame specimens subjected to vertical load only featured a beam collapse mechanism in which significant plastic deformations were observed at the mid-span of the beam (i.e. the position where the vertical load was applied) and adjacent to the beam-to-column connections, as displayed in Figure 6.18. The maximum measured tensile strain occurred at the bottom flange adjacent to the mid-span of the beam (as obtained from the strain gauge readings of either SG8 or SG10, as illustrated in Figure 6.2), and reached about 8.5% and 6.9% for specimens HSS-I-65×116-V and HSS-I-80×136-V, respectively, as listed in Table 6.3, all far in excess of the yield strain of 0.37% measured from the tensile coupon tests. Large tensile strains were also measured at the outer flanges of the columns adjacent to the beam-to-column connections (i.e. SG3 at Section 2-2 and SG13 at Section 7-7, as illustrated in Figure 6.2), with maximum tensile strains attained up to 3.1% and 2.0% (again, well in excess of the yield strain) for specimens HSS-I-65×116-V and HSS-I-80×136-V, respectively, as listed in Table 6.3. Note that the maximum measured tensile strains from the pair of sections located adjacent to the beam-to-column connections (i.e. Section 2-2 and Section 3-3, and Section 6-6 and Section 7-7 as illustrated in Figure 6.2) as well as the mid-span of the beam (i.e. Section 4-4 and Section 5-5 as illustrated in Figure 6.2) are listed in Table 6.3, where the lower values are shown in brackets.

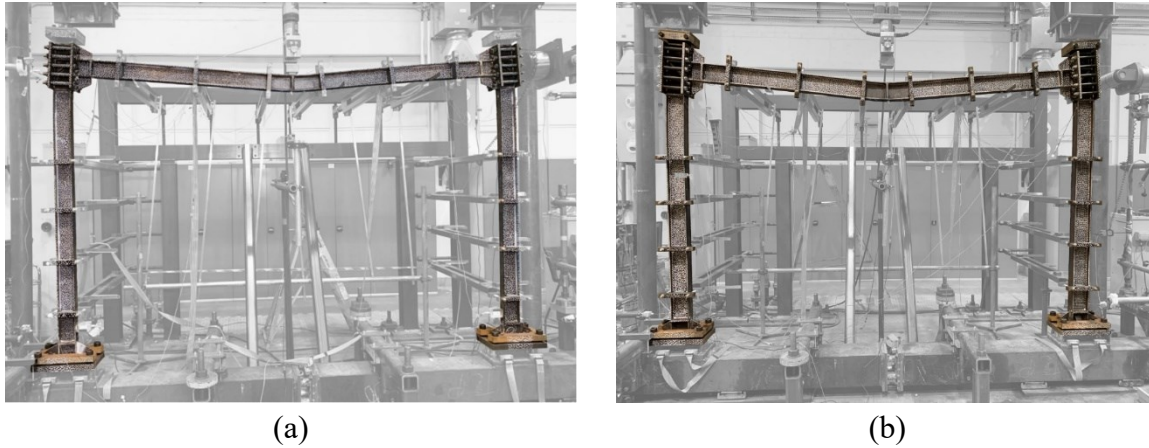


Figure 6.18: Failure modes of tested frame specimens subjected to vertical load only (a) HSS-I-65×116-V; and (b) HSS-I-80×136-V.

Table 6.3: Summary of maximum measured tensile strains and the corresponding locations.

Series	Frame label	Largest tensile strain		Second largest tensile strain		Third largest tensile strain		Fourth largest tensile strain	
		Location	Value (%)	Location	Value (%)	Location	Value (%)	Location	Value (%)
Series 1	HSS-I-65×116-V	SG10 (SG8)	8.51 (5.97)	SG13 (SG11)	3.07 (0.36)	SG3 (SG5)	1.86 (0.33)	-	-
	HSS-I-65×116-H	SG15	5.41	SG2	4.94	SG12 (SG14)	1.38 (0.49)	SG3 (SG5)	1.13 (0.97)
	HSS-I-65×116-V&H-1	SG2	3.82	SG3 (SG5)	3.11 (1.17)	SG15	2.87	SG10 (SG8)	2.27 (1.89)
	HSS-I-65×116-V&H-2	SG10 (SG8)	5.60 (2.26)	SG3 (SG5)	4.82 (0.99)	SG2	4.29	SG15	2.62
Series 2	HSS-I-80×136-V	SG8 (SG10)	6.86 (5.27)	SG3 (SG5)	2.04 (0.60)	SG13 (SG11)	1.76 (0.49)	-	-
	HSS-I-80×136-H	SG15	6.90	SG2	4.33	SG5 (SG3)	2.21 (0.87)	SG12 (SG14)	1.93 (0.43)
	HSS-I-80×136-V&H-1	SG10 (SG8)	5.90 (2.40)	SG2	4.12	SG5 (SG3)	2.83 (2.43)	SG15	2.58
	HSS-I-80×136-V&H-2	SG10 (SG8)	6.38 (4.68)	SG3 (SG5)	4.62 (1.43)	SG2	2.09	SG15	0.51

6.4.2.2 Frame specimens subjected to horizontal load only

The test specimens subjected to horizontal load only failed due to material yielding, the spread of plasticity and the formation of a sway plastic collapse mechanism, as shown in Figure 6.19 and discussed below. The maximum tensile strains were recorded from the strain gauges located adjacent to the base of the columns (i.e. SG2 at Section 1-1 and SG15 at Section 8-8,

as illustrated in Figure 6.2), with the maximum tensile strain measured from SG15 being slightly larger than that from SG2, reaching approximately 5.4% and 7.0% for frame specimens HSS-I-65×116-H and HSS-I-80×136-H, respectively, as given in Table 6.3. The measured maximum tensile strains at the locations adjacent to the beam-to-column connections (i.e. Sections 2-2, 3-3, 6-6 and 7-7, as illustrated in Figure 6.2) were all beyond the measured yield strain of 0.37%, indicating yielding had occurred at these positions prior to failure. The maximum tensile strains in the beam sections adjacent to the beam-to-column connections (i.e. SG5 at Section 3-3 and SG12 at Section 6-6, as illustrated in Figure 6.2) were generally greater than those recorded in the column sections adjacent to the corresponding beam-to-column connections (i.e. SG3 at Section 2-2 and SG14 at Section 7-7, as illustrated in Figure 6.2); the maximum measured tensile strains from SG5 were approximately 1.0% and 2.2% for frame specimens HSS-I-65×116-H and HSS-I-80×136-H, respectively, while the maximum measured tensile strains from SG12 reached about 1.4% and 1.9% for frame specimens HSS-I-65×116-H and HSS-I-80×136-H, respectively, as tabulated in Table 6.3. The attainment of high strains adjacent to the column bases and to the beam-to-column connections signifies the development of plastic hinges and the formation of the sway plastic collapse mechanism.

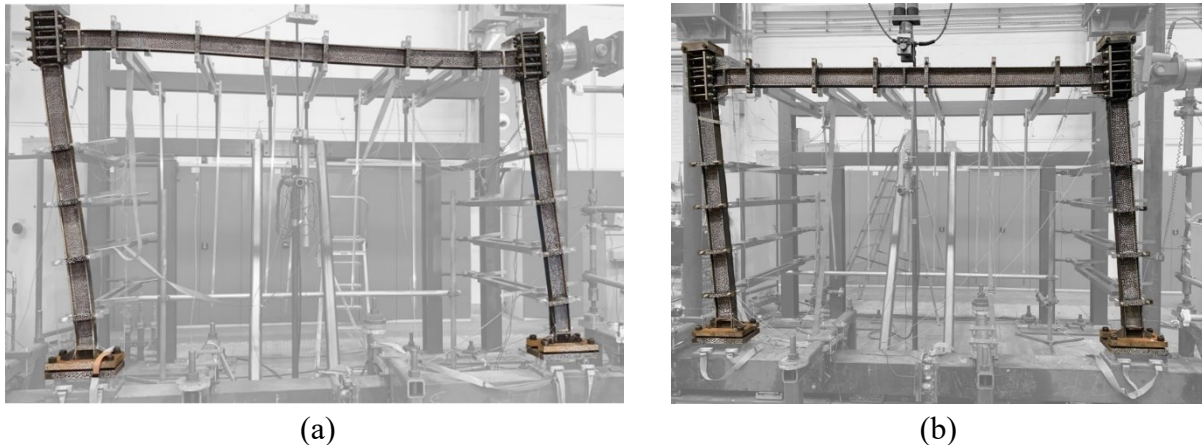


Figure 6.19: Failure modes of tested frame specimens subjected to horizontal load only (a) HSS-I-65×116-H; and (b) HSS-I-80×136-H.

6.4.2.3 Frame specimens subjected to combined loading

For each test series, two frame specimens were tested under different combinations of horizontal and vertical loads. The failure modes of these frame specimens were characterized

by a combined beam and sway plastic collapse mechanism, as shown in Figure 6.20 and Figure 6.21, but with different hinge formation sequences owing to the different loading conditions, as given in Table 6.2. The tensile strain at the bottom flange adjacent to the mid-span of the beam for frame specimen HSS-I-65×116-V&H-1 (taken as the average strain from the pair of SG8 and SG10, illustrated in Figure 6.2) reached 0.29% when the predetermined vertical load $V_{\text{test,max}}$, as given in Table 6.2, was applied, which was lower than the measured yield strain of 0.37% indicating that the stresses within the section at the mid-span of the beam remained relatively low prior to the introduction of the horizontal load. However, for the other three frame specimens, first yield occurred at the mid-span of the beam prior to the horizontal load being applied, with the maximum average tensile strains (from the pair of SG8 and SG10) being 0.7%, 0.9% and 1.7% for frame specimens HSS-I-65×116-V&H-2, HSS-I-80×136-V&H-1 and HSS-I-80×136-V&H-2, respectively, when the corresponding predetermined vertical load $V_{\text{test,max}}$ was reached. For all four frame specimens subjected to combined loading, different degrees of plastic deformation, as reflected by the maximum tensile strains reported in Table 6.3 and shown in the failure modes of Figure 6.20 and Figure 6.21, occurred at the mid-span of the beam, the column bases and adjacent to the left-hand beam-to-column connection, while the sections adjacent to the right-hand beam-to-column connection remained elastic, since they experienced relatively small levels of bending moment and compression in the combined load cases. The observed order of yielding was generally consistent with the anticipated sequence of plastic hinges determined from a first order plastic analysis.



Figure 6.20: Failure modes of tested frame specimens subjected to the first combination of horizontal and vertical loads (a) HSS-I-65×116-V&H-1; and (b) HSS-I-80×136-V&H-1.



Figure 6.21: Failure modes of tested frame specimens subjected to the second combination of horizontal and vertical loads (a) HSS-I-65×116-V&H-2; and (b) HSS-I-80×136-V&H-2.

6.4.3 Load versus displacement curves

The load versus displacement curves are presented in Figures 6.22-6.24 for the frame specimens subjected to vertical load only, horizontal load only and combined loading, respectively. The sequence of the formation of the plastic hinges (PH) and the locations of the formed plastic hinges are also indicated in Figures 6.22-6.24. Considering the amount of required rotation capacity ($R_{\text{req}} = 2.5$) for performing plastic design in frames (Bartsch et al., 2021a), a plastic hinge is deemed to have formed when the maximum tensile strain within the cross-section reaches three times the yield strain (i.e. $3\varepsilon_y = 1.11\%$) in the subsequent analyses.

6.4.3.1 Frame specimens subjected to vertical load only

The relationships between the vertical applied load and the vertical displacement at the mid-span of the beam for the frame specimens subjected to vertical load only are plotted in Figure 6.22. The initial stiffness of frame HSS-I-65×116-V was lower than that of frame HSS-I-80×136-V due to the smaller cross-section size, and the ultimate resistance was about 40% lower compared to that of HSS-I-80×136-V, as shown in Table 6.2.

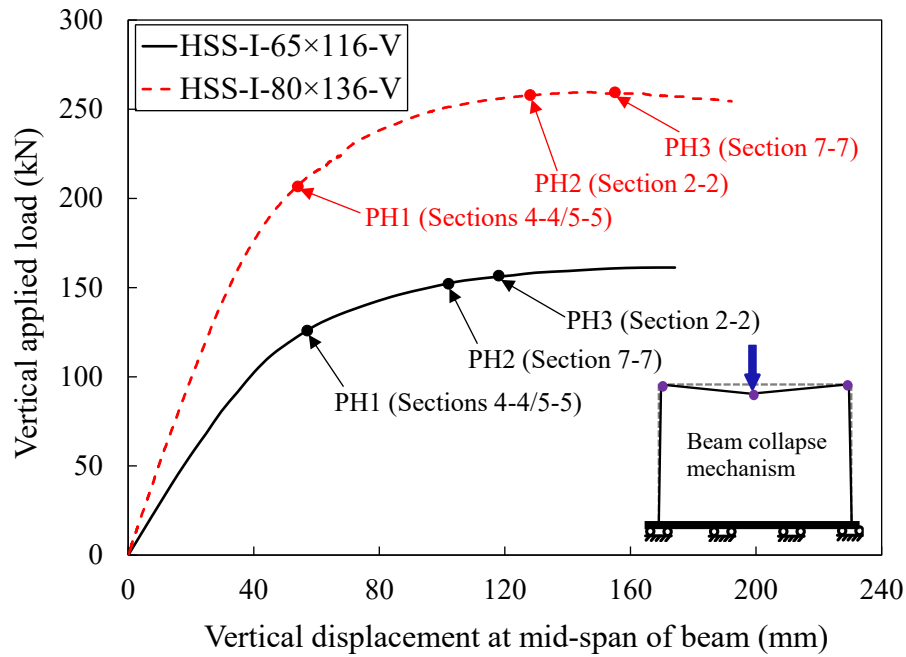


Figure 6.22: Vertical load versus mid-span displacement curves and plastic hinge positions for frame specimens subjected to vertical load only.

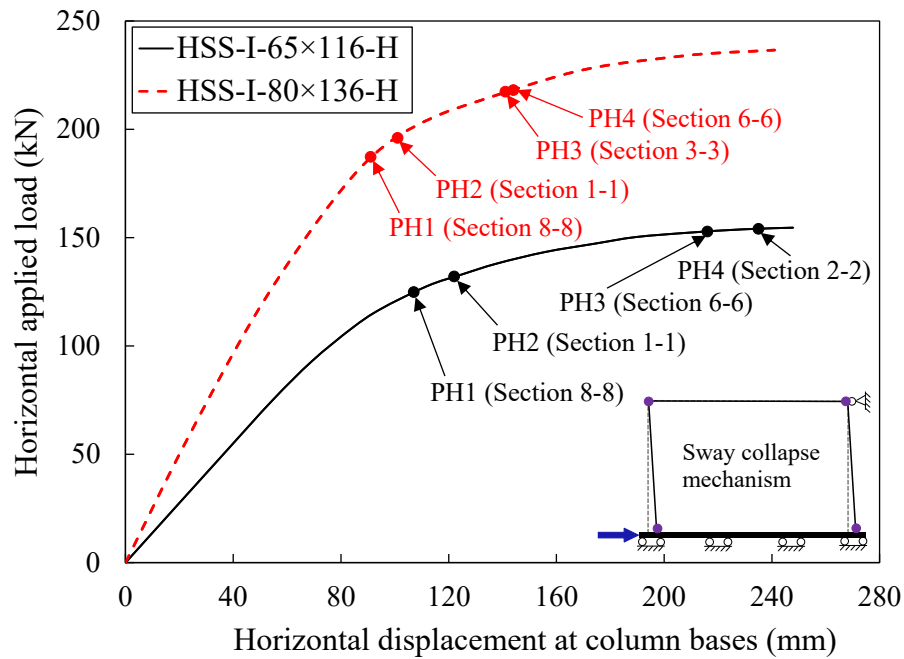


Figure 6.23: Horizontal load versus horizontal displacement curves and plastic hinge positions for frame specimens subjected to horizontal load only.

6.4.3.2 Frame specimens subjected to horizontal load only

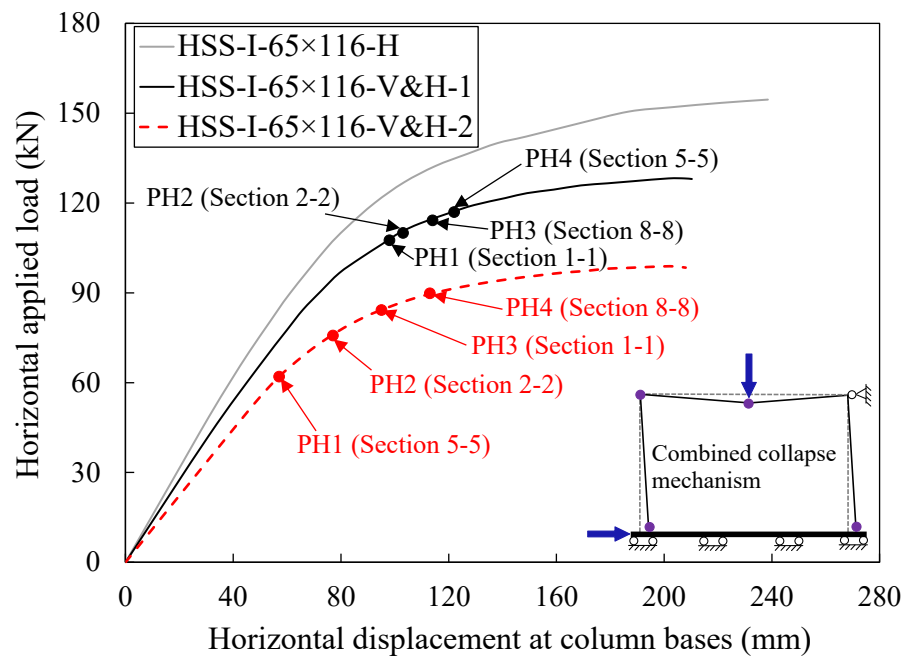
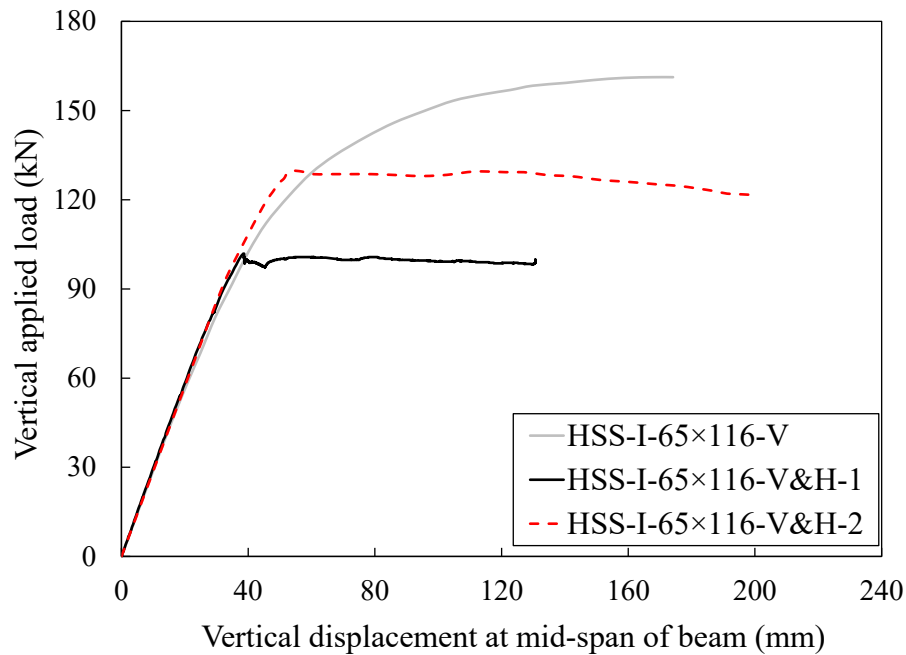
Figure 6.23 shows the responses of the frame specimens subjected to horizontal load only in

terms of the horizontal applied load versus the horizontal displacement at the column bases. Note that, for all frame specimens subjected to horizontal load, the difference between the horizontal applied load and the corresponding horizontal reaction load measured by the load cell placed within the horizontal support system was typically less than 5%. As shown in Figure 6.23, frame specimen HSS-I-65×116-H, with smaller cross-section dimensions, had a lower initial stiffness and ultimate resistance compared to those of specimen HSS-I-80×136-H.

6.4.3.3 Frame specimens subjected to combined loading

Both the vertical and horizontal responses of the frame specimens subjected to combined vertical and horizontal loads are plotted in Figure 6.24. The load-displacement curves for the frame specimens within each series subjected to vertical load or horizontal load only are also plotted in Figure 6.24 for comparison purposes. As shown in Figure 6.24, during the vertical loading step, which was always first, the vertical applied load versus the vertical displacement curves for the frame specimens subjected to combined loading conditions follow those of the corresponding frame specimens under vertical load only. The vertical load in the first combined loading case of each test series (i.e. HSS-I-65×116-V&H-1 and HSS-I-80×136-V&H-1) was relatively small, thus the vertical response of the frame specimens under the first combined loading cases was almost linear during the vertical loading step; however, the vertical load in the second combined loading case of each test series (i.e. HSS-I-65×116-V&H-2 and HSS-I-80×136-V&H-2) was large enough to result in yielding at the mid-span of the beam prior to the introduction of the horizontal load. During the horizontal loading step, the vertical load remained constant while the vertical displacement at the mid-span of the beam grew as the horizontal load increased, as shown in Figure 6.24. The presence of the vertical load for the frame specimens subjected to combined loading conditions led to a degradation of both strength and stiffness compared to those of the corresponding frame specimens subjected to horizontal load only, with the larger vertical load (i.e. the second combined loading case of each test series) resulting in greater loss of resistance and stiffness, as shown in Figure 6.24. Owing to the relatively high level of vertical load $V_{\text{test,max}}$ for frame specimen HSS-I-80×136-V&H-2, the first plastic hinge at the mid-span of the beam (Section 5-5) formed when the horizontal load had reached only about 40% of the maximum applied horizontal load $H_{\text{test,max}}$, as indicated in Figure 6.24(b). The maximum strain at the first plastic hinge continued to increase under increasing load, allowing moment redistribution and the formation of subsequent plastic hinges.

The peak load of the system was reached prior to the full development of the final plastic hinge as judged based on the strain gauge data relative to $3\varepsilon_y$, though there was, nonetheless, clear plastification in this region; this may have been due to some unloading at the location of the first plastic hinge or in the top-left region of the frame that experienced some out-of-plane deformation.



(a)

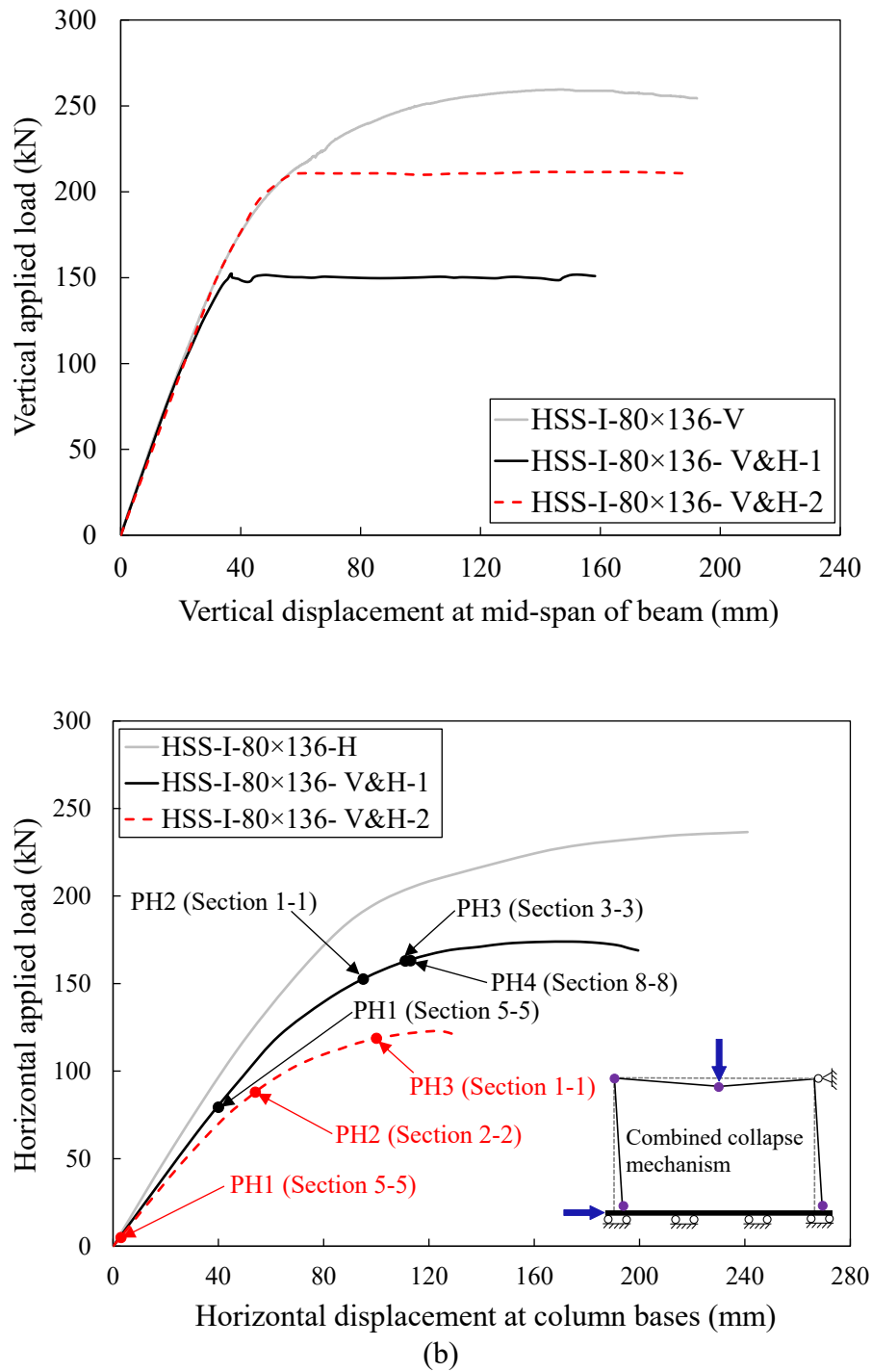


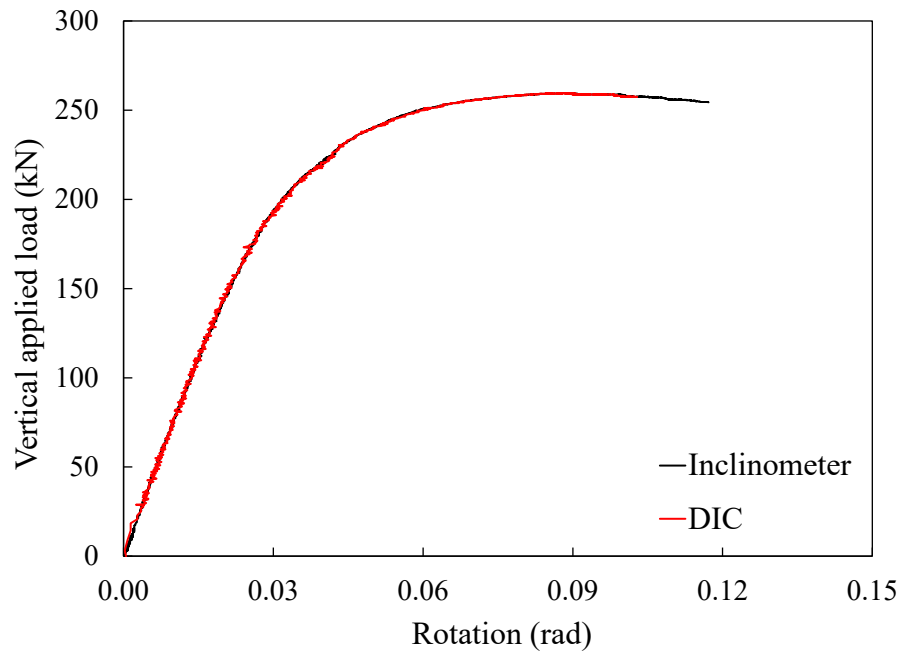
Figure 6.24: Load-displacement curves and plastic hinge positions for frame specimens subjected to combined vertical and horizontal loads (a) HSS-I-65x116-V&H-1 and HSS-I-65x116-V&H-2; and (b) HSS-I-80x136-V&H-1 and HSS-I-80x136-V&H-2.

6.4.4 DIC results and connection rotational stiffness

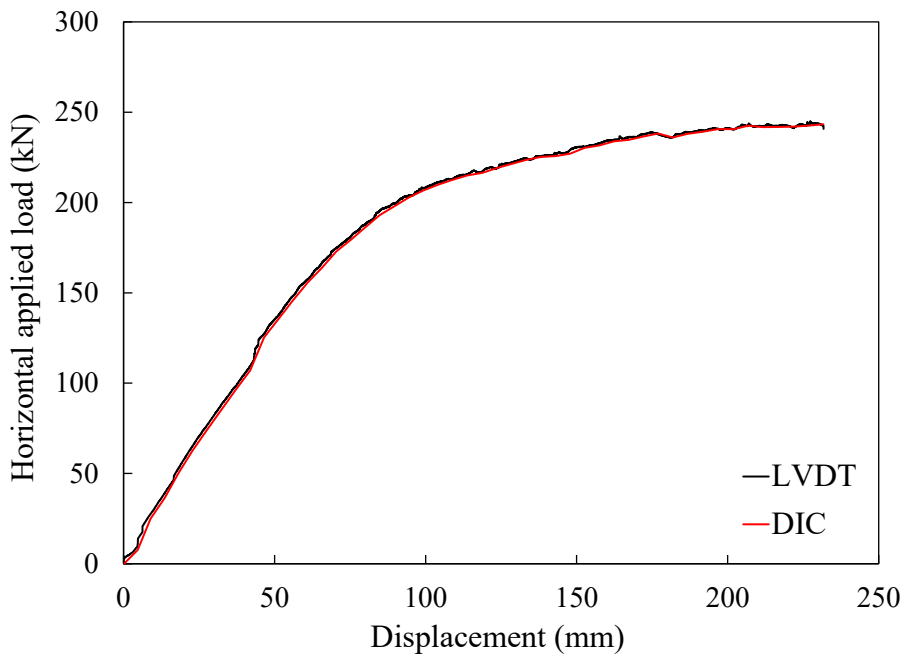
In this subsection, the DIC measurements are firstly compared with the relevant measurements from the conventional instrumentation, confirming the accuracy of the DIC technique. The DIC measured displacement fields, longitudinal strain fields and displacements at specific points of interest, which cannot be easily measured using conventional techniques, are then presented and analysed to provide a better understanding of the structural behaviour of the investigated HSS frame specimens.

6.4.4.1 DIC validation and results

To confirm the accuracy of the results obtained from the DIC system, the DIC measurements were compared with the relevant measurements from the strain gauges, inclinometers and string potentiometers. As detailed earlier, two cameras with 8 mm focal length lenses (Camera 1 and 2 indicated in Figure 6.16) were used to monitor the deformation fields of the full frame specimens; the displacement and rotation responses determined based on the images captured by the two cameras were compared to those measured from the string potentiometers and inclinometers, respectively, at the corresponding location of each frame specimen. Load-deformation histories obtained from the DIC system and the conventional instrumentation show excellent agreement, examples of which are shown in Figure 6.25 for frame specimens HSS-I-80×136-V and HSS-I-80×136-H. The histories of strain development at the most highly stressed region of each frame specimen measured from the DIC (using images taken from Camera 3 and 4 shown in Figure 6.16) and strain gauges are also compared; good agreement was also observed, as seen from the typical comparison shown in Figure 6.26. Overall, it may be concluded that the DIC system employed in the present study provides accurate and reliable measurements of both the overall deformation of the full frame specimens and the strain development histories at the most highly stressed region of each frame specimen.



(a)



(b)

Figure 6.25: Comparison of typical load-deformation responses obtained from DIC and conventional instrumentation (a) vertical applied load versus rotation measured at the position of inclinometer I3 for frame specimen HSS-I-80×136-V; and (b) horizontal applied load versus horizontal displacement at the column bases measured at the position of string potentiometer SP5 for frame specimen HSS-I-80×136-H.

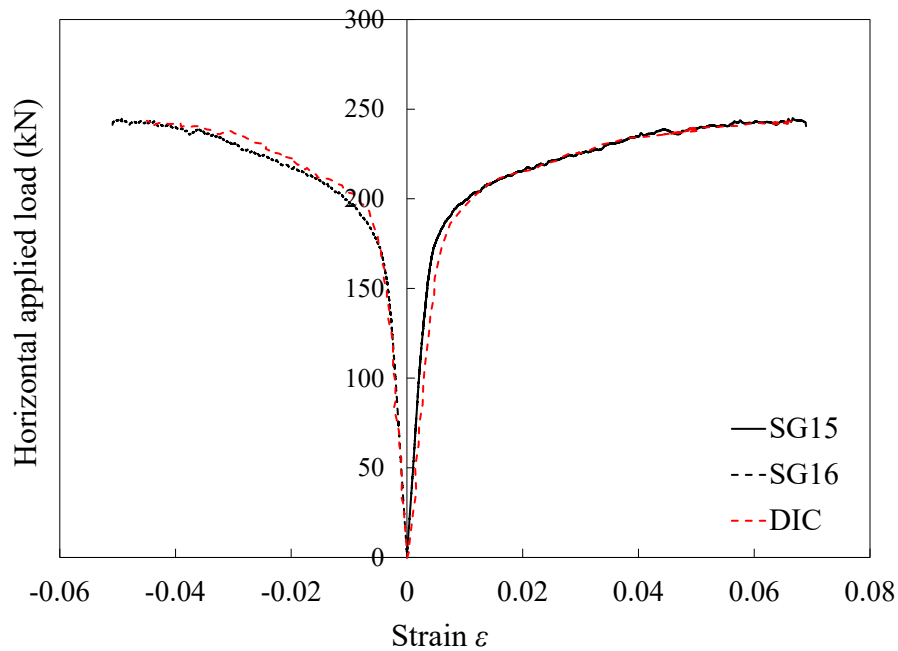
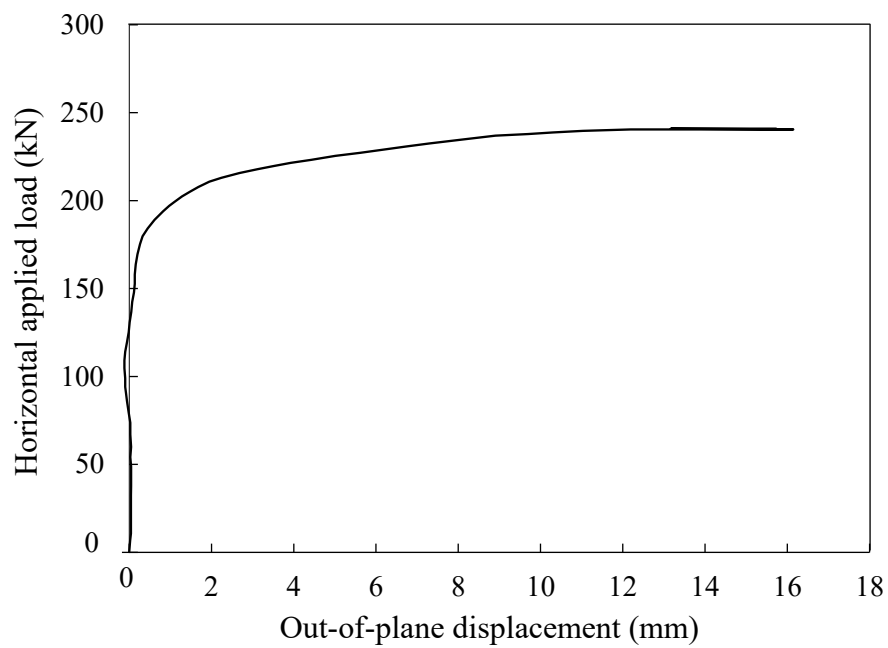


Figure 6.26: Comparison of typical histories of strain development obtained from DIC and strain gauges at the positions of SG15 and SG16 for frame specimen HSS-I-80×136-H.

The maximum out-of-plane displacement that occurred in any frame was smaller than 20 mm, confirming the generally effectiveness of the lateral restraint system. The location of the maximum out-of-plane displacement was typically in the heavily stressed top left region of the frames, where the lateral restraint spacing was slightly higher than elsewhere, although the lateral torsional buckling slenderness $\bar{\lambda}_{LT}$ of the unrestrained member length remained below 0.2. The out-of-plane displacement fields obtained from the DIC for two typical frame specimens (specimens HSS-I-80×136-H and HSS-I-65×116-V&H-1) at the maximum applied load, together with the load versus out-of-plane displacement curves at the critical region (i.e. the region where the maximum out-of-plane displacement occurred), are displayed in Figure 6.27. Note that the positive and negative out-of-plane displacements in Figure 6.27 represent movements toward and away from the DIC system, respectively, with reference to Figure 6.16. The maximum out-of-plane displacement arose in the top left region of the frames as a consequence of inelastic local buckling, accompanied with some minor influence from inelastic lateral torsional buckling, while no significant out-of-plane displacement was observed in the remaining regions. It should be noted that it is not practically possible to have zero out-of-plane deformation for structural testing at large-scale. To reduce out-of-plane deformation further, even closer lateral restraint spacing could be used, but care must be taken not to restrain local

buckling and not to provide excessive lateral restraint that does not reflect practical arrangements.



(a) HSS-I-80×136-H

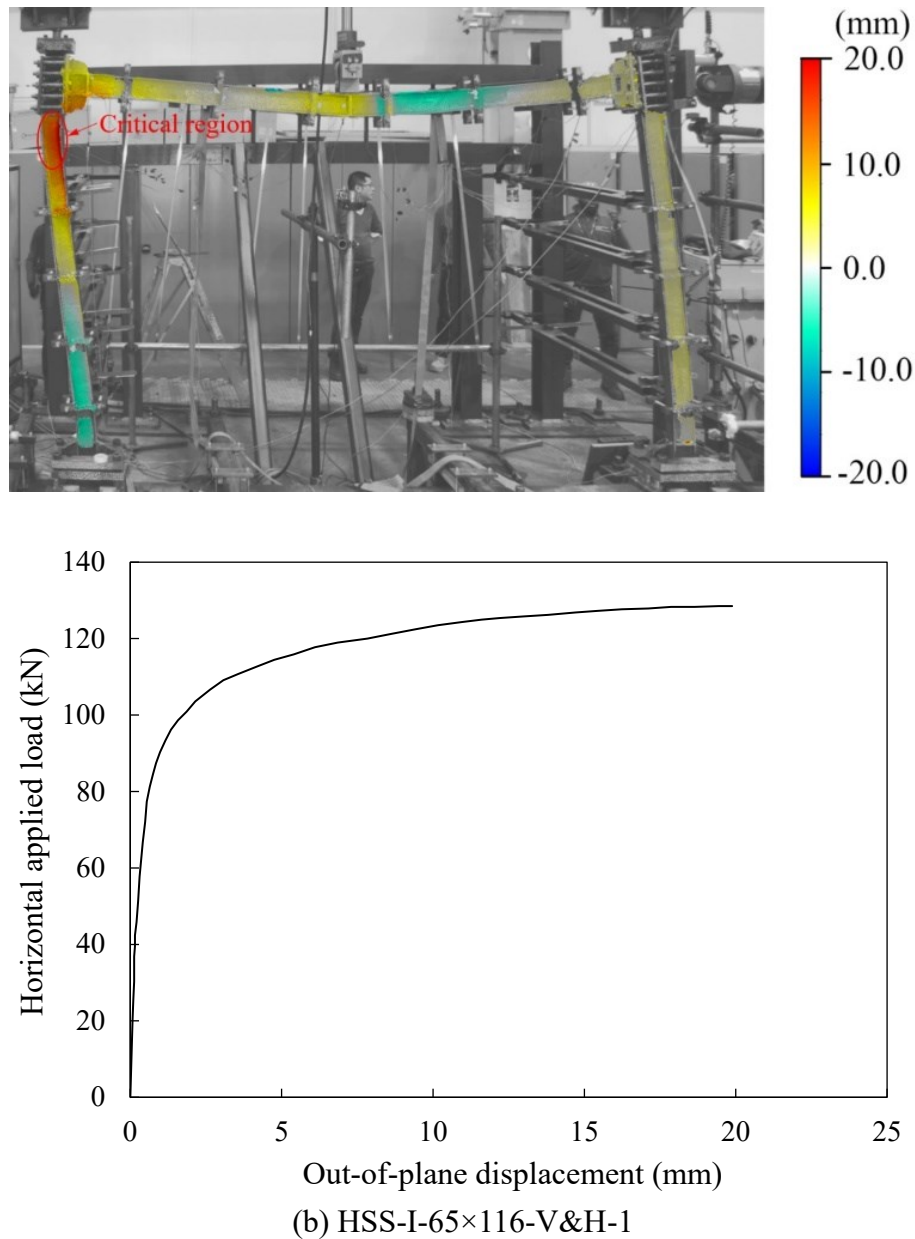


Figure 6.27: Out-of-plane displacement fields at maximum applied load and load-out-of-plane displacement curves at the critical region of three typical frame specimens obtained from DIC (a) HSS-I-80×136-H; and (b) HSS-I-65×116-V&H-1.

The longitudinal strain fields in the DIC monitored region (using images taken from Camera 3 and 4 shown in Figure 6.16) at two representative load levels: (a) the load at which the maximum strain within the DIC monitored region reached approximately the yield strain, and (b) the maximum applied load, are displayed for three typical frame specimens in Figure 6.28. The spread of plasticity with the increase in applied load in the DIC monitored region can be clearly seen by the quantitative change in the strain fields, as shown in Figure 6.28.

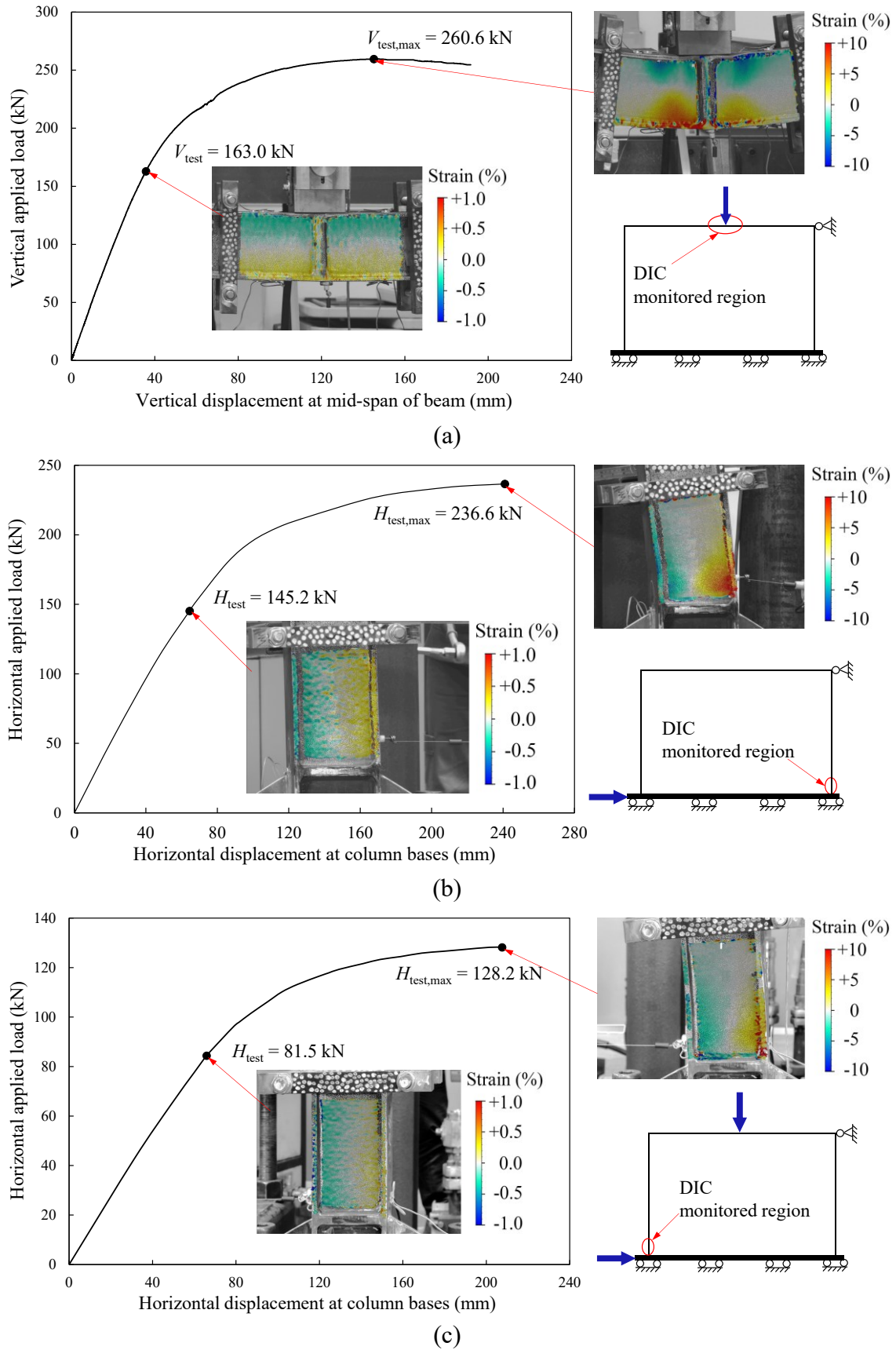


Figure 6.28: Longitudinal strain fields at different representative loads obtained from DIC for three typical frame specimens (a) HSS-I-80×136-V, (b) HSS-I-80×136-H; and (c) HSS-I-65×116-V&H-1. (Note that positive strains are tensile and negative strains are compressive).

6.4.4.2 Connection rotation stiffness

The moment-rotation characteristics of the connections in the frame specimens, including the beam-to-column connections and the column base connections, were carefully analysed using the data measured by strain gauges, inclinometers and the DIC system at relevant locations to determine their moment-rotation response.

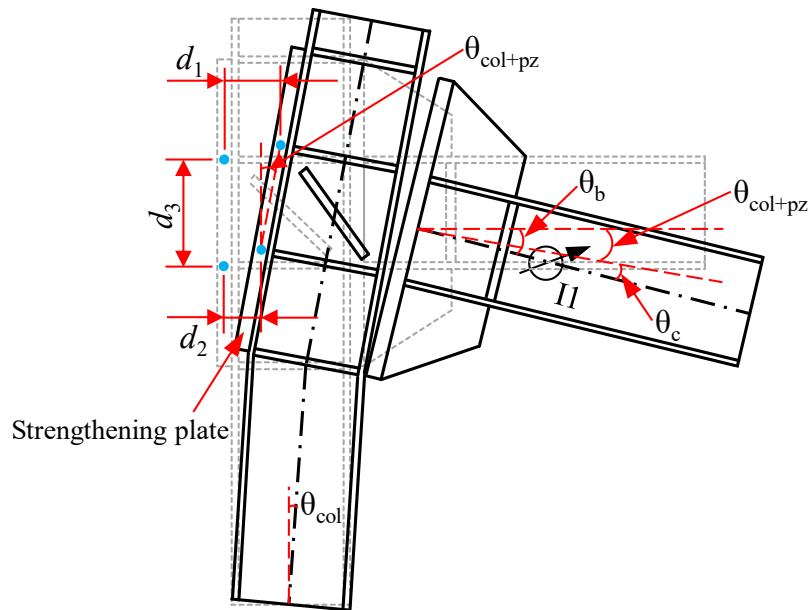


Figure 6.29: Schematic illustration of key rotations at the left-hand beam-to-column connection.

Extended end-plate joints, as shown in Figure 6.4, were used for the beam-to-column connections. The rotation of the strengthening plate θ_{col+pz} included that of both the column θ_{col} and the panel zone θ_{pz} caused by shear deformation, as schematically illustrated in Figure 6.29. The horizontal displacements at two marked regions on the strengthening plate (i.e. identified by the blue points shown in Figure 6.29) were measured by the DIC system. The rotation of the strengthening plate θ_{col+pz} was determined using Equation (6.1), where d_1 and d_2 are the DIC measured horizontal displacements at the top and bottom marked regions of the strengthening plate, respectively, and d_3 is the distance between the two marked points, as illustrated in Figure 6.29. The rotation at the beam end θ_b was measured using the inclinometer positioned at the web of the beam section close to the beam-to-column connection (i.e. using inclinometer I1 for the left-hand beam-to-column connection and inclinometer I3 for the right-

hand beam-to-column connection), as shown in Figure 6.29. The connection rotation θ_c was finally calculated by subtracting the rotation of the strengthening plate $\theta_{\text{col+pz}}$ and the rotation resulting from beam flexure $\theta_{\text{b,flex}}$ (but this was very small and is not discussed further) from the rotation at the beam end θ_b , as given in Equation (6.2).

$$\theta_{\text{col+pz}} = \arctan \left| \frac{d_1 - d_2}{d_3} \right| \quad (6.1)$$

$$\theta_c = \theta_b - \theta_{\text{col+pz}} - \theta_{\text{b,flex}} \quad (6.2)$$

In the elastic region, the bending moments at the eight monitored beam or column sections, as shown in Figure 6.2, can be determined by Equation (6.3), based on the assumptions that all the welded I-sections remained plane and the strains were linearly distributed throughout the depth of each cross-section.

$$M_i = \frac{EI |\varepsilon_{\text{max},i} - \varepsilon_{\text{min},i}|}{H} \quad (6.3)$$

In Equation (6.3), M_i is the bending moment at the i -th monitored cross-section, I and H are the second moment of area about the major axis and the depth of the cross-section, respectively, and $\varepsilon_{i,\text{max}}$ and $\varepsilon_{i,\text{min}}$ are the measured maximum and minimum strains at the extreme fibres of the i -th monitored section, respectively. The bending moment at the left-hand beam-to-column connection M_c can be determined by linear interpolation between the two calculated bending moments at Sections 3-3 and 4-4, as shown in Figure 6.30; similarly, the calculated bending moments at Sections 5-5 and 6-6 (see Figure 6.2) were used to determine the bending moment at the right-hand beam-to-column connection for each frame specimen. Figure 6.31 shows a typical moment (M_c)-rotation (θ_c) curve for the left-hand beam-to-column connection of frame specimen HSS-I-80×136-H; the moment (M_c)-rotation (θ_c) curve of the strengthening plate as well as the moment (M_c)-rotation (θ_b) curve at the beam end θ_b are also plotted in Figure 6.31. The rotational stiffness of the beam-to-column connection is taken as the slope of the regression line fitted to the elastic region of the moment (M_c)-rotation (θ_c) curve, as illustrated in Figure 6.31. The undulations in the obtained moment-rotation curves for the beam-to-column

connection, as shown in Figure 6.31, and the column-base connection, as described later and shown in Figure 6.32, resulted from noise in the strain gauge readings. The estimated beam-to-column connection rotational stiffness for each frame specimen (taken as the average value of the left- and right-hand beam-to-column connections) is reported in Table 6.4.

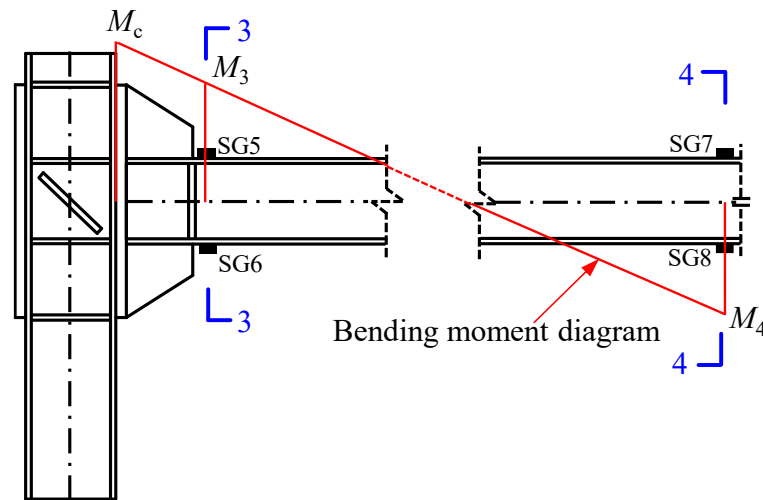


Figure 6.30: Illustration of linear interpolation to estimate bending moment at the left-hand beam-to-column connection.

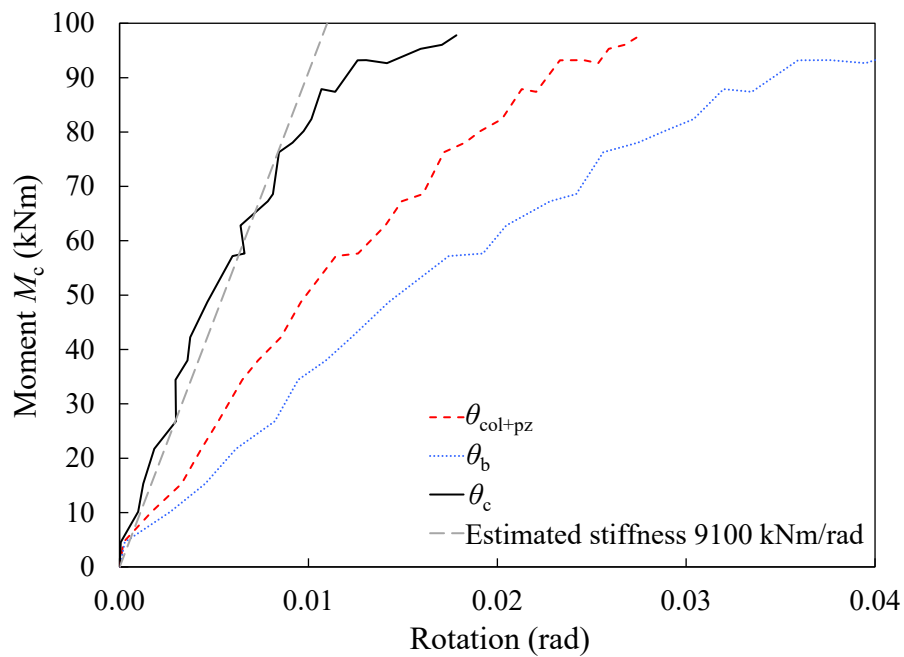


Figure 6.31: Moment-rotation curves for left-hand beam-to-column connection of frame specimen HSS-I-80×136-H.

Table 6.4: Average rotational stiffnesses of connections for the tested frames (unit: kNm/rad)

Series	Frame label	Beam-to-column connections	EN 1993-1-8 (2005) limit for rigid beam-to-column connections	Column-base connections	EN 1993-1-8 (2005) limit for rigid column-base connections
Series 1	HSS-I-65×116-V	3100	6300	3900	11600
	HSS-I-65×116-H	8400		4200	
	HSS-I-65×116-V&H-1	7900		4000	
	HSS-I-65×116-V&H-2	8200		4400	
Series 2	HSS-I-80×136-V	7100	10900	4200	20100
	HSS-I-80×136-H	8800		5800	
	HSS-I-80×136-V&H-1	8900		5900	
	HSS-I-80×136-V&H-2	8200		5900	

The rotation of each column-base connection θ_{cb} was measured using the inclinometer positioned on the web of the column section close to the column-base connection (i.e. using inclinometer I5 for the left-hand column-base connection and I6 for the right-hand column-base connection, as shown in Figure 6.2). The bending moment at each column base M_{cb} was obtained from the load cell positioned beneath the corresponding column. Figure 6.32 shows a typical moment (M_{cb})-rotation (θ_{cb}) curve for the left-hand column-base connection of frame specimen HSS-I-80×136-H. The estimated column-base connection rotational stiffness for each frame specimen (taken as the average value of the left- and right-hand column-base connections) is also reported in Table 6.4. It can be observed from Table 6.4 that the measured rotational stiffnesses for the connections with the same configurations (i.e. for each frame series) are quite similar, except for the beam-to-column connections of the first tested frame specimen HSS-I-65×116-V, which have a lower average rotational stiffness than their counterparts in the remaining frame test specimens of Series 1. This is attributed to the adoption of lower grade bolts (Grade 8.8 bolts instead of Grade 12.9 bolts for the remaining frame test specimens) and a lower applied pretension in the assembly of the beam-to-column connections in the first frame specimen HSS-I-100-65-V. Based on the joint classification rules set out in EN 1993-1-8:2005 (2005), all connections employed in the test frame specimens are semi-rigid, except for the beam-to-column connections of frames HSS-I-65×116-H, HSS-I-65×116-V&H-1 and HSS-I-65×116-V&H-2, which are rigid.

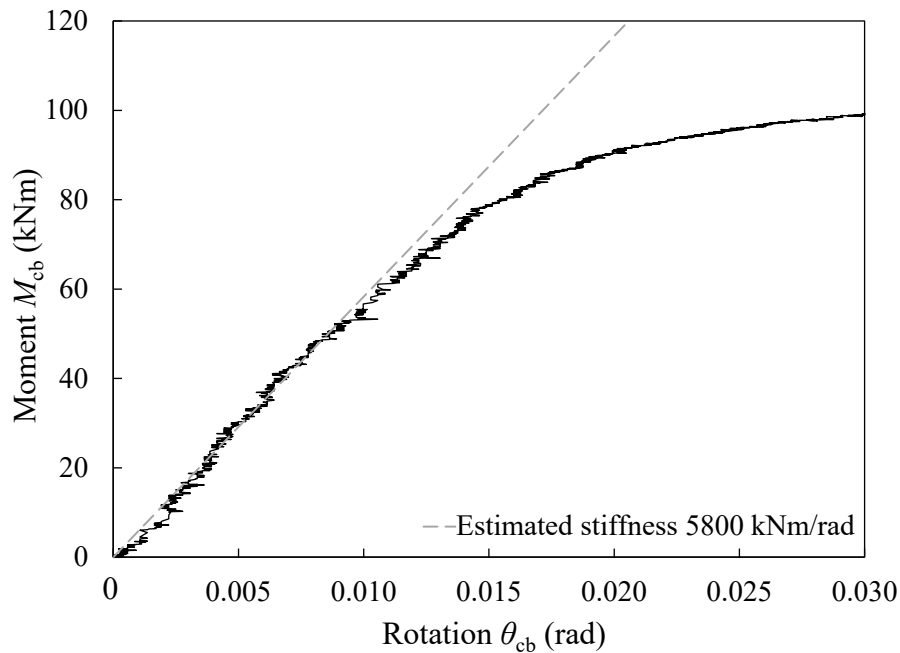


Figure 6.32: Moment-rotation curve for left-hand column-base connection of frame specimen HSS-I-80×136-H.

6.5 Analysis and discussion

The presented test results indicate that a considerable amount of inelastic moment redistribution can be achieved within HSS frames comprising compact cross-sections. However, plastic design of indeterminate HSS structures is not permitted in current design specifications (AISC 360-16, 2016; EN 1993-1-1, 2005) owing to an absence of supporting structural performance data, and the capacity of the studied HSS frames with stocky welded I-sections is limited to the load level at which the plastic moment resistance is first reached (i.e. the first plastic hinge forms), as determined by elastic analysis.

The ratios of the experimental ultimate load F_u , taken as the maximum applied vertical ($V_{test,max}$) or horizontal ($H_{test,max}$) load, as listed in Table 6.2, to the first order first plastic hinge load F_{1st_hinge} are plotted against the elastic critical load factor α_{cr} for all the tested frame specimens in Figure 6.33. The elastic critical load factor α_{cr} indicates the susceptibility of a steel frame to the global second order sway effects and is defined as the ratio of the elastic critical load F_{cr} that causes frame buckling in an in-plane global (sway) mode to the applied design load F_d . The second order effects due to the global geometrical nonlinearity are deemed to be

sufficiently small to be ignored when $\alpha_{cr} \geq 10$ according to EN 1993-1-12:2007 (2007), the limit of which is also plotted in Figure 6.33. For each frame, the value of α_{cr} was determined using linear buckling analysis of the frame under the loading corresponding to the ultimate stage of the test (Arrayago et al., 2020b; Fieber et al., 2020; Walport et al., 2019); the α_{cr} of all tested frame specimens are greater than 10, indicating that the second order effects may be neglected. Thus, first order plastic analysis was performed to examine the effect of the inelastic moment redistribution in the tested HSS frame structures. The values of F_{1st_hinge} was calculated by means of first order plastic analysis (i.e. materially nonlinear analysis MNA) (Arrayago et al., 2020b), assuming proportional horizontal-vertical loading in the ratio of $H_{test,max}/V_{test,max}$ for the frame specimens subjected to combined loading. The semi-rigid connections of the tested frame specimens were considered in the analyses by incorporating spring elements, with their rotational stiffness taken as the corresponding values measured from the frame experiments as summarized in Table 6.4, into the FE models; the modelling approach has been successfully employed by (Arrayago et al., 2020b). It can be observed from Figure 6.33 that the F_u/F_{1st_hinge} ratios ranged from 1.15 to 1.39, indicating that the current first plastic hinge (i.e. elastic) design approach (AISC 360-16, 2016; EN 1993-1-1, 2005) provides overly conservative resistance predictions for HSS frames with compact cross-sections due to the preclusion of the inelastic moment redistribution. The ratios of F_u/F_{1st_hinge} for the HSS-I-65×116×8×8 frames are generally higher than those of the HSS-I-80×136×8×8 frames; this is attributed to the more compact nature of the former cross-section (with the nominal flange width-to-thickness ratio being 2.86 for HSS-I-65×116×8×8 and 3.80 for HSS-I-80×136×8×8 and the nominal web width-to-thickness ratio being 11.10 for HSS-I-65×116×8×8 and 13.60 for HSS-I-80×136×8×8) having higher rotation capacity and hence a higher degree of inelastic moment redistribution. Hence, as expected, for frame specimens with the same geometries and boundary conditions and subjected to the same loading configuration, those made up with stockier cross-sections would possess higher ductility than those with more slender cross-sections. The comparisons demonstrate the available ductility and highlight the potential for plastic design of HSS structures with compact cross-sections. It should be noted that the frame over-strength F_u/F_{1st_hinge} depends on several parameters, such as the frame geometry, boundary conditions, loading conditions and material properties; further investigation on this topic is deemed necessary through a numerical parametric study.

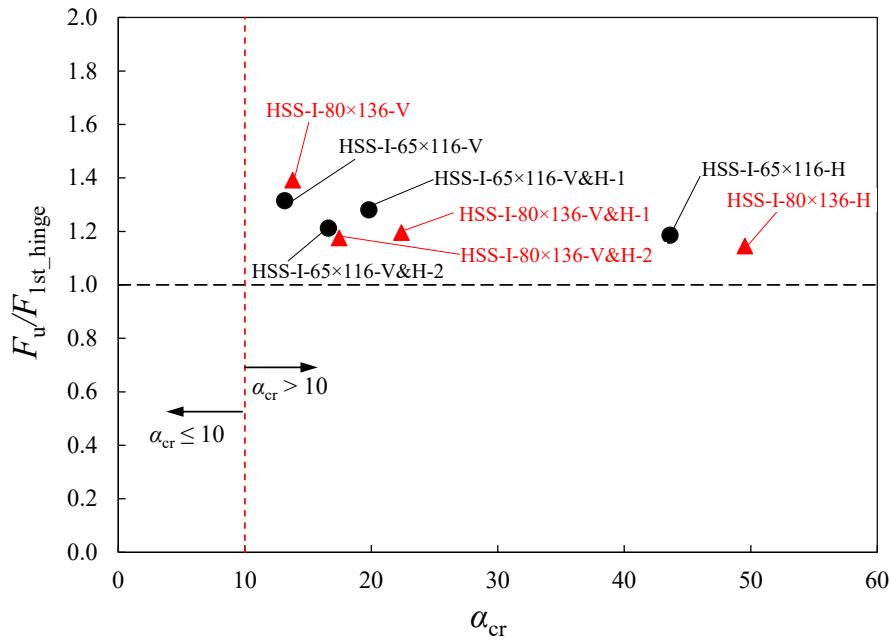


Figure 6.33: Ratios of F_u/F_{1st_hinge} versus α_{cr} for the tested frame specimens.

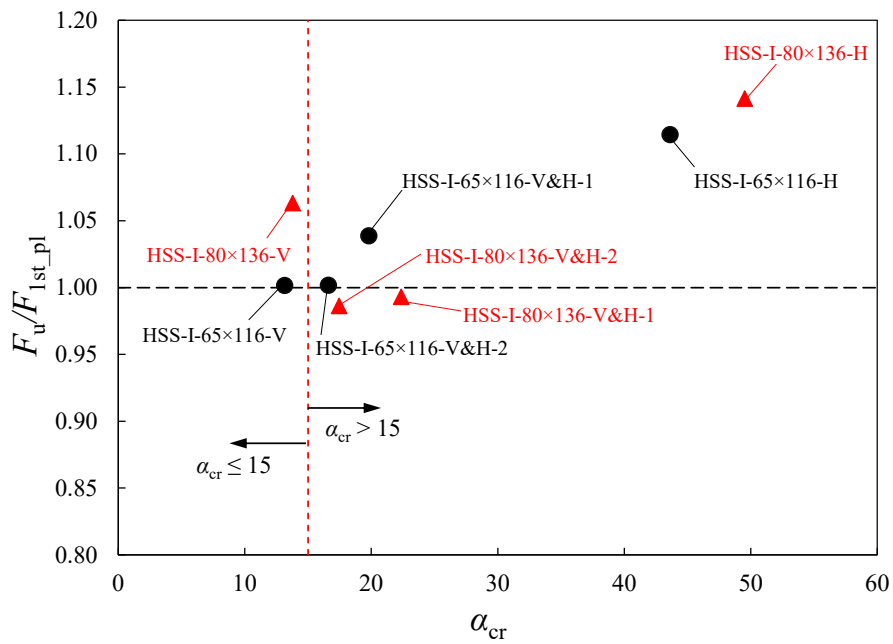


Figure 6.34: Ratios of F_u/F_{1st_pl} versus α_{cr} for the tested frame specimens.

The ratios of the experimental ultimate load F_u to the first order plastic load F_{1st_pl} obtained from a first order plastic analysis are plotted against the elastic critical load factor α_{cr} for all the tested frame specimens in Figure 6.34. For frames designed using plastic analysis, a higher limit of $\alpha_{cr} \geq 15$, beyond which second order effects may be neglected, is defined in EN 1993-

1-1:2005 (2005), taking account of the fact that plastically-designed steel frames are more susceptible to second order effects due to their reduced stiffness caused by the spread of plasticity and the formation of plastic hinges (Walport et al., 2019a, 2019b). It may be concluded from Figure 6.34 that the current plastic design methods set out in EN 1993-1-1:2005 (2005) are generally suitable for HSS frames, though the limit of $\alpha_{cr} = 15$ above which second order effects can be ignored for plastic analysis is somewhat crude and results in slightly unconservative results for frame specimens HSS-I-80×136-V&H-1 and HSS-I-80×136-V&H-2. Further investigation on the applicability of the limit of $\alpha_{cr} = 15$ to HSS frames is on-going.

6.6 Conclusions

A comprehensive experimental investigation into the structural behaviour of HSS frames made of S690 welded I-sections has been presented in this chapter. A total of eight two-dimensional, single bay, single storey, unbraced rectangular HSS frames were tested. The experimental setup, including the implementation of different loading schemes, the use of a bespoke restraint system to prevent out-of-plane instability and the application of specially fabricated load cells to measure the reaction forces and bending moments at the supports of the frame specimens, has been described. The supporting system was carefully designed to ensure the proper functionality of the experimental setup, which is considered to be essential in carrying out large-scale structural testing. It is hoped that the test setup presented in this article will help steer future research in structural testing, particularly in the case of steel frame testing at large-scales. Moreover, the advantages of using the DIC technique in large-scale structural testing have been demonstrated in the present study; the DIC was shown to be able to provide accurate, reliable and detailed information concerning the fields of displacement and strain both within regions of particular interest and for the overall tested system. The ultimate collapse loads, load-deformation characteristics and failure modes of the frames, together with the rotational stiffnesses of the connections, have been carefully presented and analysed. The experimental results indicated that, despite the lower ductility and strain hardening of HSS relative to normal strength steel (NSS), HSS frames with stocky cross-sections have the ability to form plastic hinges and achieve a considerable amount of inelastic moment redistribution. In addition, it was found that the current elastic design approach as employed in EN 1993-1-12:2007 (2007) provides overly conservative resistance predictions for HSS frames made of Class 1 S690

welded I-sections, while the plastic design approach set out in EN 1993-1-1:2005 (2005) for normal strength steel that allows for moment redistribution at the global system level results in more accurate resistance predictions. The available ductility and potential for plastic design of HSS structures has been highlighted and will be examined further in the future work. The presented experimental investigation of HSS frames at the structural system level have allowed the assessment of the interaction between plastic collapse and global instability, and provide benchmark results that are suitable for the validation of finite element models and the development of system-level design approaches.

Chapter 7

Design of HSS welded I-section members by GMNIA with strain limits

7.1. Introduction

The focus of this chapter is to extend a recently developed framework of design by advanced analysis (Fieber et al., 2019a; Walport et al., 2021) to both NSS and HSS welded I-section members subjected to various loading scenarios. Benchmark shell FE models were firstly developed and validated against existing test data on welded I-section members subjected to bending, compression and combined loading. The validated shell FE models were then employed to generate additional benchmark data which were used to assess the accuracy of the proposed GMNIA + CSM design framework, relative to the conventional Eurocode design methods. The key features of the proposed GMNIA + CSM framework, including the CSM strain limits that define the maximum attainable strain prior to cross-section failure, the strain averaging approach that allows for the influence of moment gradient and the shear resistance check that accounts for the influence of high shear forces, are presented and discussed. The assessments reveal that the proposed GMNIA + CSM design approach offers more accurate and consistent resistance predictions for both NSS and HSS welded I-section members subjected to different loading scenarios compared to the current EC3 design approaches.

7.2 Benchmark shell finite element modelling

7.2.1 Basic modelling assumptions

In this section, shell FE models are used to generate extensive benchmark data for the assessment of the proposed GMNIA + CSM design framework for NSS and HSS welded I-sections subjected to different loading scenarios. The benchmark shell FE models were created using the four-noded shell element with reduced integration S4R in the FE software ABAQUS (2018). The notation of the geometries of modelled welded I-sections is shown in Figure 7.1, where B is the flange width, H is the outer section depth, t_f is the flange thickness, t_w is the web thickness, t_{weld} is the weld leg length, b_f is the clear width of the outstand flange equal to $(B - t_w)/2 - t_{\text{weld}}$, and h_w is the clear height of the web equal to $H - 2t_f - 2t_{\text{weld}}$. The flange width of each modelled I-section was subdivided into an even number of elements with the mesh size approximately equal to $(B+H)/40$. The selected mesh size was also uniformly applied along the web depth and member length, resulting in an element aspect ratio close to unity for elements assigned to the clear width and height of the flange and web plates.

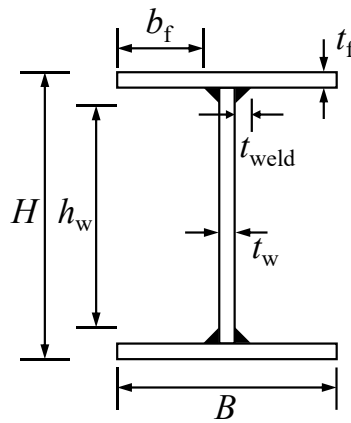


Figure 7.1: Notation for modelled welded I-sections

Both global and local geometric imperfections were incorporated into each shell FE model by modifying the nodal coordinates of the perfect geometry. Global imperfections were assumed in the shape of a half sinusoidal wave with the imperfection magnitude equal to $L/1000$, where L is the member length, and were introduced into the FE models subjected to compression and

combined compression and bending. Local imperfections were introduced into the FE models subjected to all types of loading scenarios as a series of sinusoidal waves with the imperfection magnitude defined following the recommendations given in EN 1993-1-5 (2006), as graphically illustrated in Figure 7.2. Specifically, when the flange plate is more susceptible to local buckling (i.e. the elastic local buckling stress of the flange plate $\sigma_{cr,f}$ is lower than that of the web plate $\sigma_{cr,w}$), the magnitude of the local flange imperfection was taken equal to 1/50 of its clear flange width b_f ; otherwise, the magnitude of the local web imperfection was taken equal to 1/200 of its clear web depth h_w . The local imperfection magnitude of the corresponding non-critical plated element was then determined by assuming that the web-flange intersections remained at 90°. The half-wavelength of the local imperfections was defined approximately equal to the elastic local buckling half-wavelength of the modelled I-section in compression $L_{b,cs}$. The analytical expressions proposed by Fieber et al. (2019) were used to determine the $L_{b,cs}$ values in the present study, and the obtained value of $L_{b,cs}$ was adjusted to fit the nearest odd number of half-wavelengths to the member length.

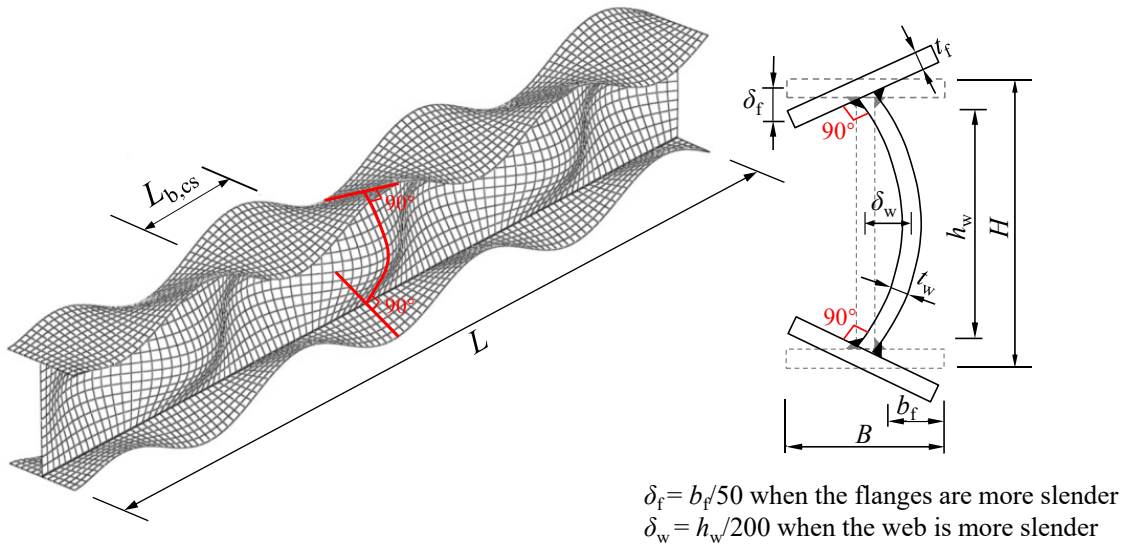


Figure 7.2: Definition of local geometric imperfections along the member length.

The measured stress-strain curves from previous experimental programmes (Chapter 3 and 4; Ban et al., 2013b; Deng et al., 2021; Li et al., 2016; Ma et al., 2017; Sun et al., 2019; Yang et al., 2018) were used for the validation of the developed shell FE models, while for the

subsequent parametric studies, the bilinear plus nonlinear hardening stress-strain model developed by Yun and Gardner (2017) for hot-rolled steel was used to generate engineering stress-strain curves for all the steel grades considered in this research. The Poisson's ratio ν was taken equal to 0.3 in the elastic range and 0.5 in the plastic range. The engineering stress-strain curves were transformed into true stress-logarithmic plastic strain curves before inputting into the FE models. The residual stress pattern proposed in Section 3.2 for welded I-sections with varying steel grades was introduced into the FE models as an initial stress condition, as illustrated in Figure 3.2. A preliminary analysis step was defined to achieve equilibration of the residual stresses, and a subsequent analysis step was set up for the application of external loadings by means of the modified Riks method, enabling the full load-deformation responses of the FE models to be captured.

Suitable boundary conditions and external loadings were applied at reference points which were coupled with nodes within the corresponding end sections or loading regions through rigid body constraints. For members subjected to bending alone, transverse stiffeners were modelled and attached to the cross-sections at which the loading was applied to effectively prevent localised web crippling failure under concentrated transverse force. Note that for members bending/buckling about the major axis, the out-of-plane displacement degree of freedom was restrained along the flange centrelines at adequate intervals to prevent lateral torsional buckling.

7.2.2 Validation

The developed shell FE models were validated against a total of 48 experimental results on HSS welded I-section members subjected to different loading scenarios, including beams in three- and four-point bending (Chapter 4; Sun et al., 2019), columns buckling about the major and the minor axes (Chapter 3; Ban et al., 2013; Deng et al., 2021; Li et al., 2016) and beam-columns subjected to combined compression and minor axis bending (Ma et al., 2017b; Yang et al., 2018). The measured geometries and material properties were used in the shell FE models for validation purposes. The fillet welds of the I-sections were explicitly modelled using five web elements at each web-flange interaction with different increased thicknesses (Chapter 3 and 4). The material properties of the fillet welds, if not available in the literature, were

assumed to be equal to those of the web plate. Comparisons of the numerically obtained ultimate capacities $R_{u,FE}$ against the corresponding test results $R_{u,test}$ are summarised in Table 7.1, including the mean and coefficient of variation (COV) values of the $R_{u,FE}/R_{u,test}$ ratios for welded I-section members subjected to different loading conditions. It can be seen from Table 7.1 that the developed shell FE models are able to provide ultimate capacities very close to the corresponding test results. The numerically obtained load-deformation curves also agree well with the corresponding experimental curves, as shown in Figure 7.3 for typical beam specimens and Figure 7.4 for typical column and beam-column specimens. Hence, it can be concluded that the developed shell FE models can accurately simulate the structural response of welded I-section members and are able to provide reliable benchmark data for the assessment of different design approaches as discussed in the following sections.

Table 7.1: Comparisons of test and numerical results for welded I-section members subjected to different loading scenarios

Member type	Loading scenario	Steel grade	No. of tests	$R_{u,FE}/R_{u,test}$		References
				Mean	COV	
Beam	Three/four-point bending	S690	12	1.02	0.02	Chapter 4; Sun et al., 2019
Column	Buckling about major axis/minor axis	Q460GJ, S690, Q960	20	0.96	0.07	Chapter 3; Ban et al., 2013; Deng et al., 2021; Li et al., 2016
Beam-column	Compression + minor axis bending	Q460GJ, S690	16	0.98	0.04	Ma et al., 2017; Yang et al., 2018

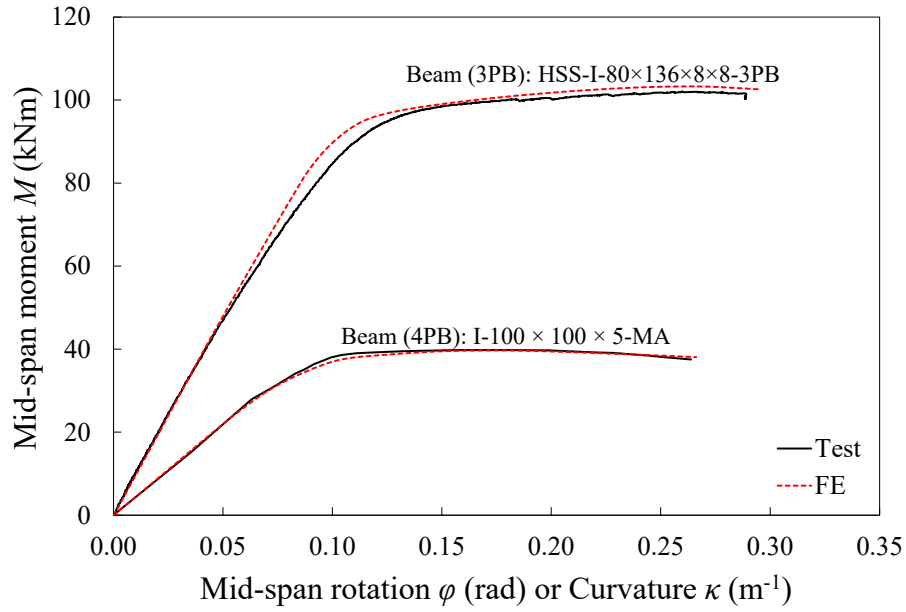


Figure 7.3: Comparisons of typical test and FE load-deformation curves for welded I-section beams made of S690 steel subjected to three-point bending (3PB) (Chapter 3) and four-point bending (4PB) (Sun et al., 2019).

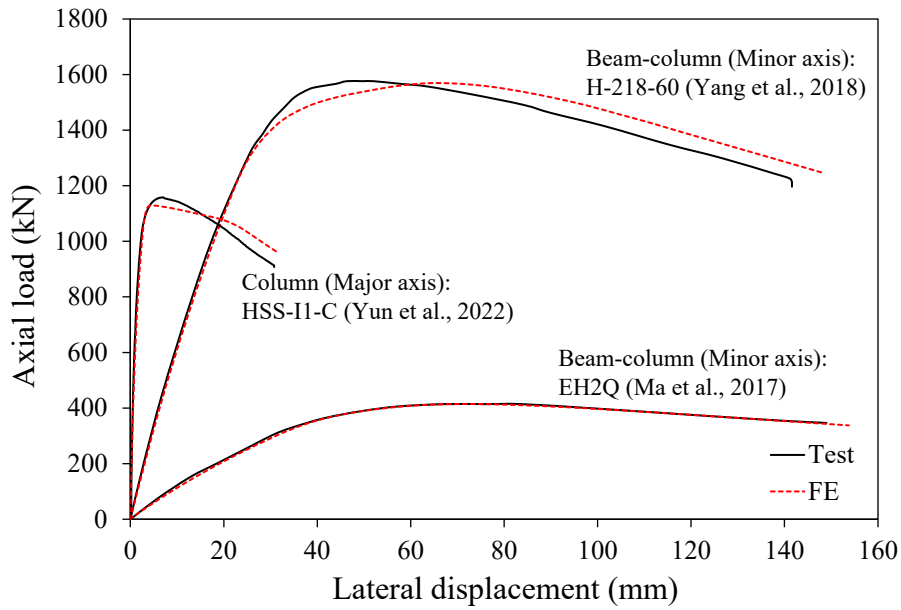


Figure 7.4: Comparisons of typical test and FE load-deformation curves for welded I-section columns (Chapter 3) and beam-columns (Ma et al., 2017b; Yang et al., 2018) made of S690 steel.

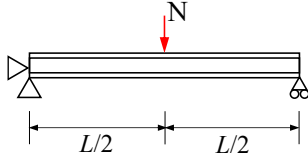
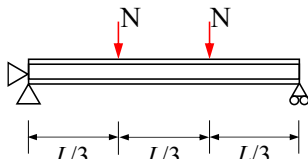
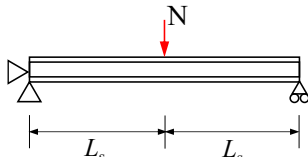
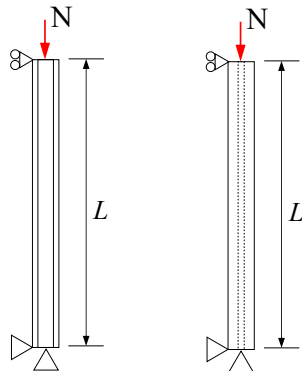
7.2.3 Parametric study

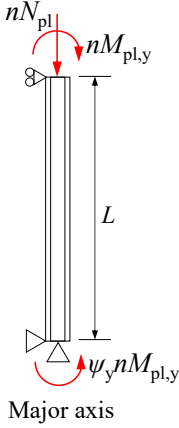
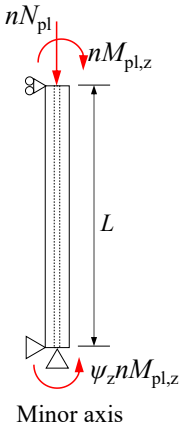
Upon validation of the shell FE models, a comprehensive parametric study was performed to create extensive benchmark data for the evaluation of different design approaches. All the modelled welded I-sections in the parametric study had a fixed flange width B of 100 mm while three outer section depths H of 100 mm, 150 mm, and 200 mm were considered, resulting in cross-section aspect ratios equal to 1.0, 1.5 and 2.0, respectively. Different values of flange and web thicknesses were selected to generate a broad spectrum of cross-section slenderness values $\bar{\lambda}_p$ varying from 0.2 to 1.0 (i.e. covering cross-sections from Class 1 to Class 4 according to prEN 1993-1-1 (2019)). For members subjected to bending, the web thickness t_w of each modelled I-section was taken equal to 0.6 times the corresponding flange thickness t_f (i.e. $t_w = 0.6t_f$), which represents a most common web-to-flange thickness ratio for standard hot-rolled UB and IPE profiles. For members subjected to compression or combined loading, the flange and web thicknesses of each modelled I-section were determined such that their corresponding plate slendernesses in compression were approximately equal (i.e. the flange plate slenderness $\bar{\lambda}_{p,f} \approx$ the web plate slenderness $\bar{\lambda}_{p,w}$), thereby minimising the influence of element interaction between the flange and the web on the ultimate resistances of these structural members. Note that the fillet welds of the I-sections were ignored throughout the parametric study. Two steel grades (S355 and S690) were employed in the parametric study. The stress-strain curves of the two adopted steel grades were represented by the bilinear plus nonlinear material model proposed by Yun and Gardner (2017). Key mechanical properties (E, f_y and f_u) of the considered steel grades are provided in Table 7.2. A summary of the load cases and key varied parameters considered in the parametric study are illustrated in Table 7.3, where L_s is the shear span of the 3PB specimens taken equal to $L/2$, $\bar{\lambda}_y$ and $\bar{\lambda}_z$ are the non-dimensional member slenderness about the major and minor axes, respectively, ψ_y and ψ_z are the end moment ratios about the major and minor axes, respectively, and n is the design load ratio equal to N_{Ed}/N_{pl} .

Table 7.2: Key mechanical properties for S355 and S690 steels employed in the present study.

Steel grade	E	f_y	f_u
	N/mm ²	N/mm ²	N/mm ²
S355	210000	355	490
S690	210000	690	770

Table 7.3: Summary of key load cases and the varied parameters considered in the parametric study for the assessment of different design methods.

Load case	No. of FE models	Varied parameters
Three-point major axis bending 	S355: 200 S690: 200	Cross-section: Cross-sections are selected such that the corresponding cross-section local slendernesses $\bar{\lambda}_p$ vary from 0.2 to 1.0. Member: Member lengths L are consistently set equal to 15 times outer section depths H .
and four-point major axis bending 		
Three-point major axis bending (Shear check) 	S355: 144 S690: 144	Cross-section: 18 Class 1-2 cross-sections are selected. ($B \times H \times t_f \times t_w$ in mm) S355: 100 × 100 × 6 × 3.6 100 × 100 × 9.5 × 5.7 100 × 100 × 13 × 7.8 100 × 150 × 6 × 3.6 100 × 150 × 9.5 × 5.7 100 × 150 × 13 × 7.8 100 × 200 × 6 × 3.6 100 × 200 × 9.5 × 5.7 100 × 200 × 13 × 7.8 S690: 100 × 100 × 8.5 × 5.1 100 × 100 × 12 × 7.2 100 × 100 × 15 × 9 100 × 150 × 8.5 × 5.1 100 × 150 × 12 × 7.2 100 × 150 × 15 × 9 100 × 200 × 8.5 × 5.1 100 × 200 × 12 × 7.2 100 × 200 × 15 × 9 Member: The selected shear spans L_s vary from $L_{b,cs}$ to $12L_{b,cs}$.
Compression with buckling about:  Major axis Minor axis	S355: 216 S690: 210	Cross-section: Cross-sections are selected such that the corresponding cross-section local slendernesses $\bar{\lambda}_p$ vary from 0.2 to 1.0. Member: For each considered cross-section and for each axis of buckling, three member lengths L are selected such that the corresponding non-dimensional member slendernesses $\bar{\lambda}_y$ (or $\bar{\lambda}_z$) are equal to 0.5, 1.1 and 1.7.

Load case	No. of FE models	Varied parameters
Combined compression and bending with buckling about:	S355: 1980 S690: 1980	Cross-section: 24 Class 1-4 cross-sections are selected. ($B \times H \times t_f \times t_w$ in mm) S355: $100 \times 100 \times 4 \times 2.6$ $100 \times 100 \times 5.6 \times 3.4$ $100 \times 100 \times 6.1 \times 3.7$ $100 \times 100 \times 8 \times 4.7$ $100 \times 150 \times 4 \times 4$ $100 \times 150 \times 5.6 \times 5.4$ $100 \times 150 \times 6.1 \times 5.9$ $100 \times 150 \times 8 \times 7.7$ $100 \times 200 \times 4 \times 5.4$ $100 \times 200 \times 5.6 \times 7.5$ $100 \times 200 \times 6.1 \times 8.2$ $100 \times 200 \times 8 \times 10.9$ S690: $100 \times 100 \times 5.8 \times 3.5$ $100 \times 100 \times 8 \times 4.7$ $100 \times 100 \times 8.3 \times 4.8$ $100 \times 100 \times 16 \times 7.8$ $100 \times 150 \times 5.8 \times 5.7$ $100 \times 150 \times 7.8 \times 7.5$ $100 \times 150 \times 8.3 \times 7.9$ $100 \times 150 \times 16 \times 14.5$ $100 \times 200 \times 5.8 \times 7.8$ $100 \times 200 \times 7.5 \times 10.2$ $100 \times 200 \times 8.3 \times 11.3$ $100 \times 200 \times 16 \times 23$
		Member: For each considered cross-section and for each axis of buckling, three member lengths L are selected such that the corresponding non-dimensional member slendernesses $\bar{\lambda}_y$ (or $\bar{\lambda}_z$) are equal to 0.5, 1.1 and 1.7. Loading: End moment ratio about the major axis $\psi_y = 1, 0, -0.5$ End moment ratio about the minor axis $\psi_z = 1, -0.5$ For each member and for each end moment ratio, a total of eleven different initial loading eccentricities e_0 are considered to achieve a wide range of loading combinations (i.e. design load ratios n vary from 0 (bending alone) to 1.0 (compression alone)).
		

7.3 Design by GMNIA using beam finite elements with strain limits

To overcome the limitations of the current EC3 design approaches, a practical method of design by advanced analysis was proposed by Fieber et al. (2019b, 2020) for steel structures. The proposed design method consists of: (1) conducting a GMNIA using beam FE models, and (2) determining the ultimate capacities of individual members or structures in accordance with either the peak load factor or the load factor at which a specific strain limit, defined by the

CSM, is attained, whichever occurs first. Extension of this method to HSS prismatic welded I-section members is addressed in this section.

7.3.1 Continuous strength method (CSM)

The continuous strength method (CSM) is a deformation-based method that replaces the concept of cross-section classification with a continuous relationship (i.e. the CSM base curve) between cross-section slenderness $\bar{\lambda}_p$ and deformation capacity (strain ε) (Gardner et al., 2023). The CSM base curve, given by Equation (7.1) for plated cross-sections, defines the limiting strain ε_{csm} that a cross-section can endure prior to reaching its ultimate capacity. In the equation, cross-section slenderness $\bar{\lambda}_p$ is equal to the square root of the yield strength f_y divided by the elastic local buckling stress of the cross-section $\sigma_{\text{cr,cs}}$, as given by Equation (7.2), ε_y is the yield strain equal to the yield strength f_y divided by the Young's modulus E , ε_u is the ultimate strain estimated as $\varepsilon_u = 0.6(1-f_u/f_y)$ but no less than 0.06 for hot-rolled steels (Yun and Gardner, 2017) where f_u is the ultimate strength and C_1 is a coefficient corresponding to the quad-linear material model developed by Yun and Gardner (2017).

$$\frac{\varepsilon_{\text{csm}}}{\varepsilon_y} = \begin{cases} \frac{0.25}{\bar{\lambda}_p^{3.6}} \leq \min\left(\Omega, \frac{C_1 \varepsilon_u}{\varepsilon_y}\right) & \text{for } \bar{\lambda}_p \leq 0.68 \\ \left(1 - \frac{0.222}{\bar{\lambda}_p^{1.05}}\right) \frac{1}{\bar{\lambda}_p^{1.05}} & \text{for } \bar{\lambda}_p > 0.68 \end{cases} \quad (7.1)$$

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{\text{cr,cs}}}} \quad (7.2)$$

Two upper bounds are applied to the strain ratio $\varepsilon_{\text{csm}}/\varepsilon_y$: the first limit of Ω is set to prevent excessive strains and defines the permissible level of plastic deformation on a project-by-project basis, with a generally recommended value of 15 according to the EN 1993-1-1 ductility requirement (EN 1993-1-1, 2005); the second limit of $C_1 \varepsilon_u/\varepsilon_y$ defines a cut-off strain to avoid over-predictions of material strength when using the CSM resistance functions developed for hand calculations (Gardner et al., 2023; Yun et al., 2018a; Yun & Gardner, 2017). It was found from experimental investigations (Bartsch et al., 2021a; Lee et al., 2013; McDermott, 1969;

Ricles et al., 1998) that brittle fractures occurred on flange plates in tension of some I-sections made of HSS, with a fracture strain ε_f of about 7-8% (approximately equal to $\varepsilon_f/\varepsilon_y = 15-18$); hence, a value of $\Omega = 10$ is suggested as the upper limit to the CSM base curve for design of HSS structures to avoid the undesirable occurrence of brittle fracture on tensile plate elements made of HSS prior to reaching design loads. To conclude, values of $\Omega = 15$ for the design of NSS structures and $\Omega = 10$ for the design of HSS structures are considered in the present study. The CSM base curves for S355 steel and S690 steel are shown in Figure 7.5.

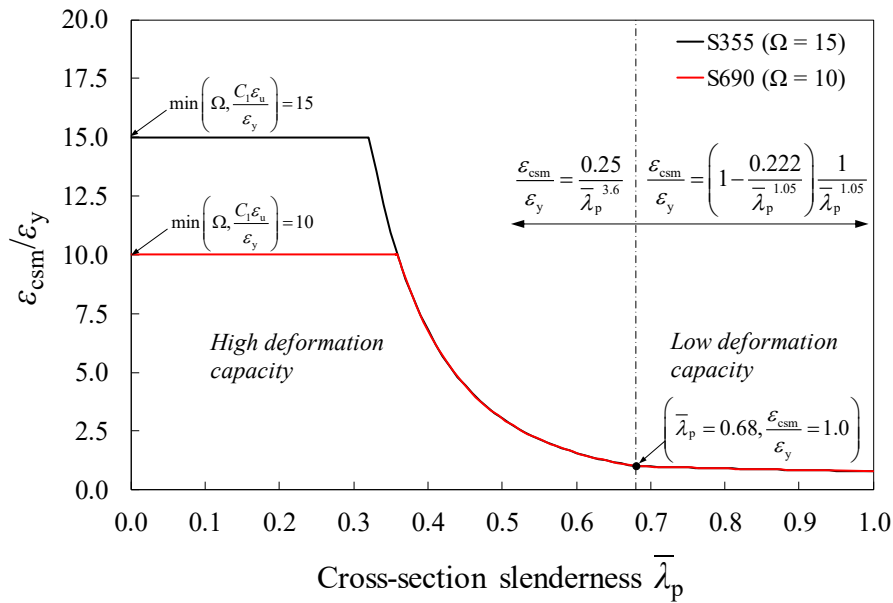
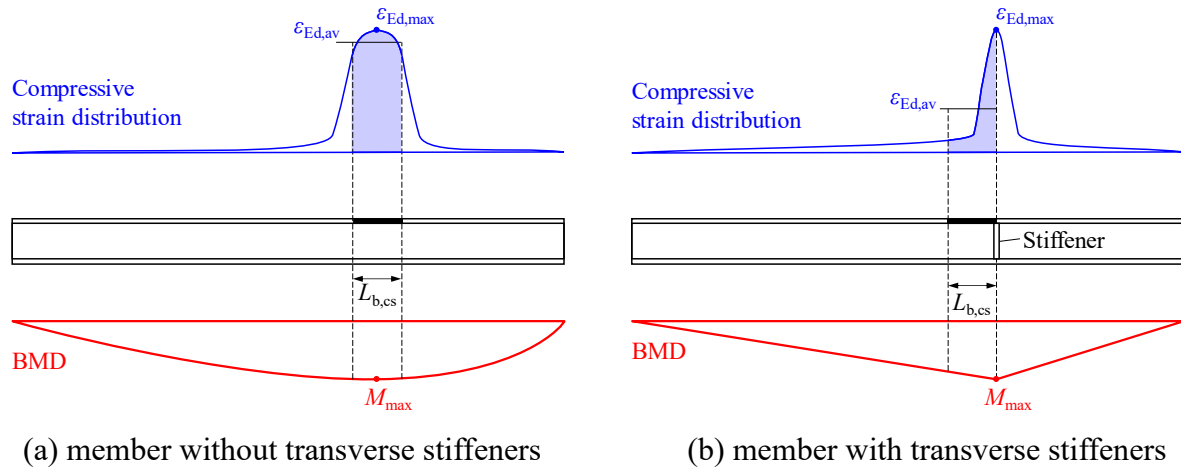


Figure 7.5: CSM base curves for S355 steel and S690 steel.

7.3.2 Strain averaging approach

Structural steel members in practice are typically subjected to varying bending moments along their lengths. Moment gradients enable the less heavily loaded cross-sections to provide some support to the critical (mostly stressed) cross-section in resisting local buckling, thereby leading to higher levels of cross-sectional capacities relative to members under uniform bending, as has been widely recognised from previous research (Chapter 4; Chan and Gardner, 2008; Wang et al., 2016). To account for this beneficial effect, a strain averaging approach was developed and introduced to the framework of design by GMNIA with CSM strain limits (Fieber et al., 2019a). The strain averaging approach requires application of the CSM strain limit ε_{csm} to

averaged compressive strains $\varepsilon_{Ed,av}$ output over a characteristic distance along the member length rather than the maximum compressive strain $\varepsilon_{Ed,max}$ of the member. The characteristic distance is taken equal to the elastic local buckling half-wavelength $L_{b,cs}$ and is determined using the analytical equations in Fieber et al. (2019b) in this study. The location of the characteristic distance corresponds to the critical cross-section, usually identified as the cross-section subjected to the maximum bending moment M_{max} , as shown in Figure 7.6(a). For uniform members with transverse stiffeners welded at the critical cross-section, the characteristic distance is located to the side of the stiffener with the shallower moment gradient, as illustrated in Figure 7.6(b). Note that only beam elements that are fully located within the characteristic length $L_{b,cs}$ are considered in the strain averaging approach.



(a) member without transverse stiffeners (b) member with transverse stiffeners
Figure 7.6: Bending moment diagrams (BMD), compressive strain distributions and typical locations of the characteristic distances ($L_{b,cs}$) for (a) members without transverse stiffeners and (b) members with transverse stiffeners.

7.3.3 Implementation of CSM strain limit to GMNIA using beam finite elements

The proposed design method is carried out by firstly performing GMNIA using beam finite elements. The beam FE models were established using the finite element software ABAQUS (2018) in this study; the method can also be implemented in any other structural analysis software that can fully capture material nonlinearity and second order effects at the member and system levels (i.e. $P - \delta$ and $P - \Delta$ effects). The 2-noded shear deformable Timoshenko beam element for open sections, referred to as B31OS in the ABAQUS element library, was

used. 121 section points were assigned to the full cross-section to ensure that the spread of plasticity through the cross-section and the distribution of initial residual stresses can be accurately simulated. The residual stresses were incorporated into the beam FE models by defining the initial stress values at the section points through the SIGINI user subroutine (ABAQUS, 2017). Global geometric imperfections were defined in the shape of a half sinusoidal wave with the magnitude equal to $L/1000$ and were modelled by directly adjusting nodal coordinates from the perfect geometry. Each beam FE model was meshed such that the element length was equal to that used in the longitudinal direction of the corresponding shell FE model. Note that the selected mesh sizes should be no greater than the relevant local buckling half-wavelength $L_{b,cs}$ of the modelled cross-section to allow for the application of the strain averaging approach. The true stress-logarithmic plastic strain curves generated based on the bilinear plus nonlinear hardening material model (Yun and Gardner, 2017) was input into the beam FE models. Pin-ended boundary conditions were applied to all cases with lateral torsional buckling effects being effectively prevented by appropriate lateral restraints. The validity of the developed beam FE models under different loading conditions has been confirmed by comparing load-deformation and load-strain histories obtained from the beam FE models against those obtained from the benchmark shell FE models.

After GMNIA using beam finite elements is conducted, the ultimate capacity of an individual member can be determined through the following procedure:

- (1) Use the analytical formulae in (Gardner et al., 2019) to calculate the elastic local buckling stress $\sigma_{cr,cs}$ and the local buckling half-wavelength $L_{b,cs}$ for the constituent cross-section according to the first-order internal force and moment distribution along the member. Based on the elastic local buckling stress $\sigma_{cr,cs}$, the cross-section slenderness $\bar{\lambda}_p$ is obtained using Equation (7.2). The CSM limiting strain ε_{csm} is then determined based on the cross-section slenderness $\bar{\lambda}_p$ using Equation (7.1).
- (2) If the member is subjected to a moment gradient, the strain averaging approach should be applied and the averaged compressive strain outputs $\varepsilon_{Ed,av}$ are checked against the CSM limiting strain ε_{csm} ; otherwise, the maximum compressive strain outputs $\varepsilon_{Ed,max}$ of the member are checked against the CSM limiting strain ε_{csm} . Note that in all cases considered in this study, the compressive strain outputs and the corresponding CSM limiting strain are both compatibly determined at the centreline of the wall thickness, as suggested by Fieber et al. (2019b).

(3) Finally, the ultimate capacity of the member is determined in accordance with either the peak load factor α_{peak} or the CSM load factor α_{csm} at which the CSM limiting strain is attained, whichever occurs first. If the peak load factor α_{peak} is firstly reached, the ultimate capacity is taken equal to the peak load; otherwise, the ultimate capacity is defined by the load when the CSM load factor α_{csm} is attained.

7.3.4 Influence of high shear forces

High levels of shear force have a negative influence on the bending capacity of steel members under moment gradients. To allow for this effect, a reduction factor ρ_{csm} given by Equation (7.3) is applied to the CSM strain limit ϵ_{csm} in the proposed design method. When the design shear force V_{Ed} exceeds half the plastic shear resistance of the cross-section $V_{\text{pl,Rd}}$ given by Equation (7.4), the ultimate shear and bending capacities are determined according to the attainment of the reduced CSM strain limit $\rho_{\text{csm}}\epsilon_{\text{csm}}$ when checking the averaged strain outputs $\epsilon_{\text{Ed,av}}$. In Equation (7.4), A_v is the shear area defined by $h_w t_w$ in the present study, γ_{M0} is the partial factor taken equal to 1.0 according to EN 1993-1-1 (2005, 2019). Note that the proposed design method still requires separate shear resistance and shear buckling checks in accordance with the current EC3 design rules (EN 1993-1-1, 2005, 2019; EN 1993-1-5, 2006), as recommended in (Fieber et al., 2019a; Walport et al., 2021).

$$\rho_{\text{csm}} = \begin{cases} 1 & \text{for } V_{\text{Ed}} \leq 0.5V_{\text{pl,Rd}} \\ \frac{0.5}{0.5 + (2V_{\text{Ed}}/V_{\text{pl,Rd}} - 1)^2} & \text{for } V_{\text{Ed}} > 0.5V_{\text{pl,Rd}} \end{cases} \quad (7.1)$$

$$V_{\text{pl,Rd}} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{\text{M0}}} \quad (7.2)$$

The effect of shear force on the normalised ultimate bending moment resistance M_u/M_{pl} is demonstrated in Figure 7.7, where the M_u/M_{pl} ratios of a series of S690 steel welded I-section beams subjected to three-point bending (3PB) with varying member lengths are plotted against their normalised shear span $L_s/L_{\text{b,cs}}$. As shown from Figure 7.7, for very short beams with

$L_s/L_{b,cs} < 3$, the failure becomes dominated by shear force with decreasing member length, leading to a significant reduction in ultimate bending moment capacities. The beam FE models cannot effectively capture this negative effect, providing inaccurate and unconservative bending moment capacities; the accuracy has been greatly improved in the proposed design method by applying the reduction factor ρ_{csm} to the CSM limiting strain, together with separate shear resistance checks, i.e. $V_{Ed}/V_{pl,Rd} \leq 1$. Moreover, since the beneficial effects of material strain hardening and moment gradients are rationally exploited by the CSM and the strain averaging approach, respectively, the proposed design method generally predicts more accurate, but still safe sided, bending moment capacities for bending dominated cases relative to the EC3 design approach.

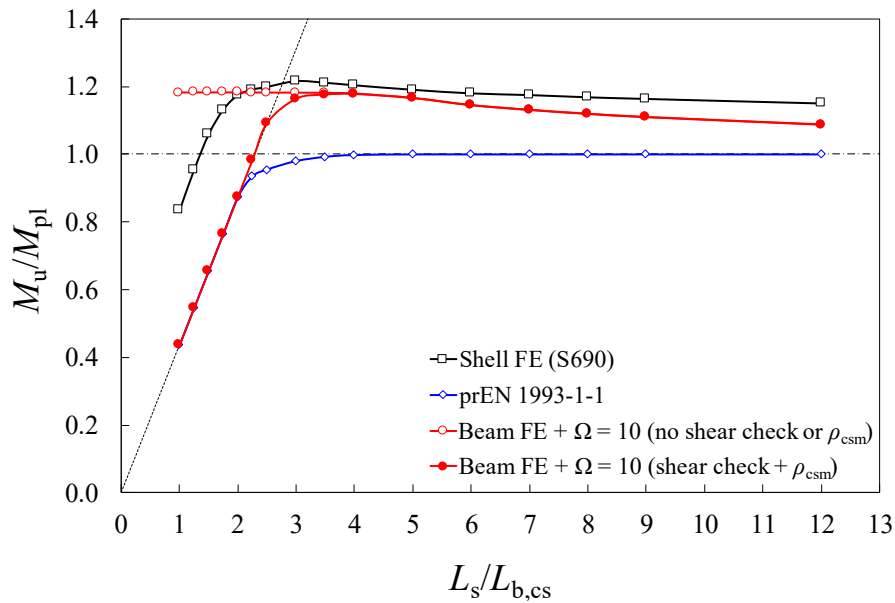


Figure 7.7: Effect of shear force on the ultimate bending moment resistance of a series of S690 steel beams (I-section shape: 100×100×15×9 in mm).

Similar conclusions can be obtained from Figures 7.8 and 7.9, which display the normalised moment-shear interaction relationships for beams made of S355 and S690 steel, respectively, where M_u and V_u are the ultimate bending and shear capacities of the cross-section, respectively. A total of 144 cases were simulated for each steel grade, considering nine different cross-section geometries and sixteen different member lengths, as introduced in Table 7.3. Table 7.4 reports the statistical analysis results of R_{shell}/R_{pred} for bending dominated cases with $V_u/V_{pl} \leq$

1.0, where R_{shell} and R_{pred} are the radial distances measured from the origin to the datapoint of the benchmark shell FE results and the datapoint predicted using the considered design method, respectively, in the normalised moment-shear diagram, as illustrated in Figure 7.10. It can be concluded that the proposed design method, with an upper limit of $\Omega = 15$ for NSS and $\Omega = 10$ for HSS, is able to provide resistance predictions that are safe sided and more accurate compared to the current EC3 design approach for bending dominated beams, with the overall mean and COV values of $R_{\text{shell}}/R_{\text{pred}}$ equal to 1.027 and 0.044, respectively.

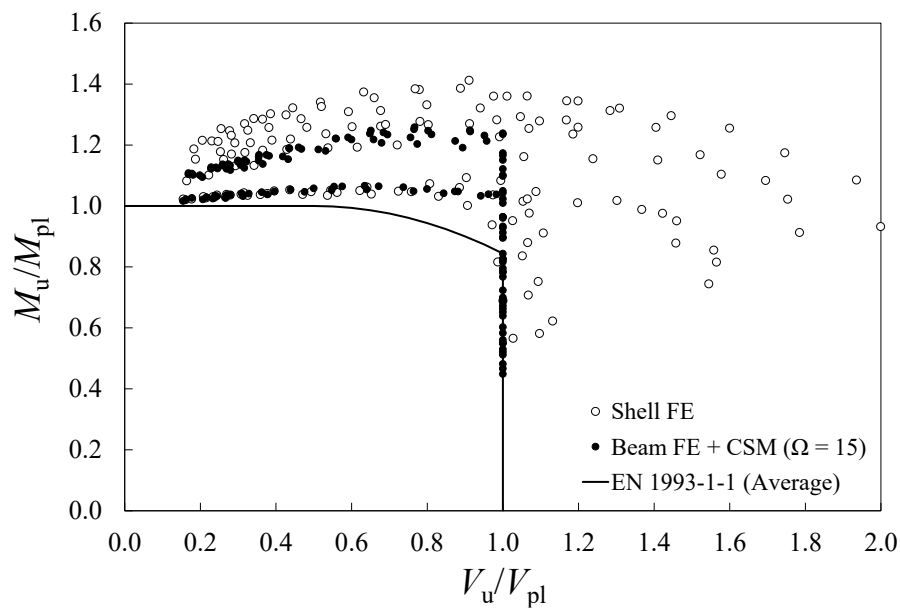


Figure 7.8: Normalised moment-shear interaction for S355 steel welded I-section beams under three-point ending predicted by shell FE benchmark modelling, beam FE modelling with CSM strain limits and EN 1993-1-1 cross-section check.

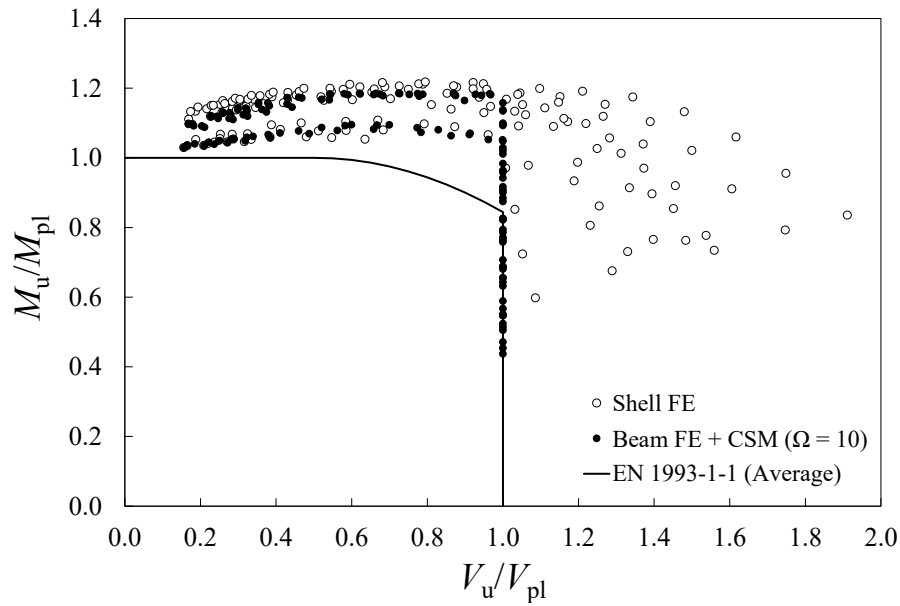


Figure 7.9: Normalised moment-shear interaction for S690 steel welded I-section beams under three-point ending predicted by shell FE benchmark modelling, beam FE modelling with CSM strain limits and EN 1993-1-1 cross-section check.

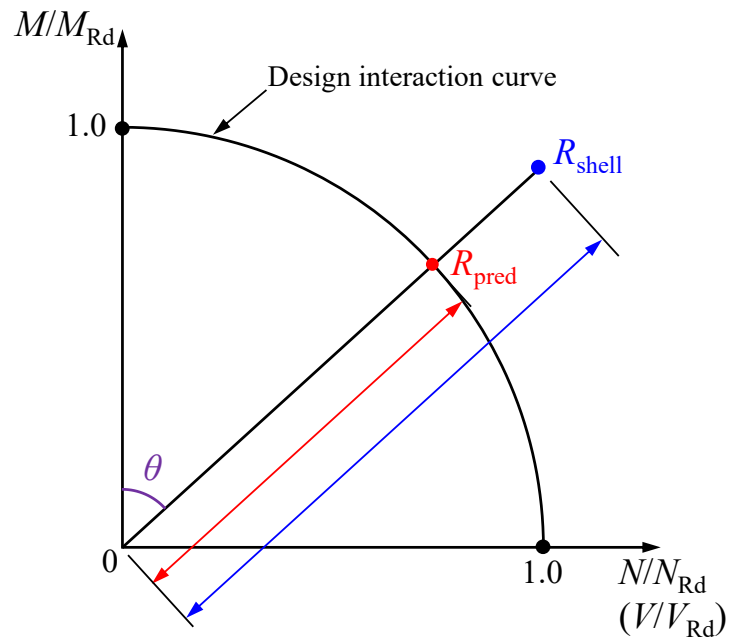


Figure 7.10: Definition of the radial distances R and the angle parameter θ in normalised M - V or M - N interaction diagram.

Table 7.4: Statistical analysis results of benchmark ultimate capacities R_{shell} against the corresponding resistance predicted using different design methods R_{pred} for beams of varying lengths with $V_u/V_{\text{pl}} < 1.0$.

Steel grade	No. of cases	Design method	$R_{\text{shell}}/R_{\text{pred}}$			
			Mean	COV	Max	Min
S690	93	EC3	1.148	0.053	1.258	1.029
		AA + CSM ($\Omega=10$)	1.015	0.020	1.070	0.971
S355	94	EC3	1.179	0.101	1.425	1.012
		AA + CSM ($\Omega=15$)	1.039	0.055	1.168	0.950
Overall	187	EC3	1.164	0.082	1.425	1.029
		AA + CSM	1.027	0.044	1.168	0.971

7.4 Evaluation of the proposed design method

The accuracy of the proposed method for the design of HSS welded I-section members subjected to bending, compression, and combined compression and uniaxial bending is assessed in this section; comparisons are made against the current EC3 design approaches as well as the NSS counterparts. The assessment was carried out by comparing the benchmark shell FE results R_{shell} against the ultimate capacities predicted using the considered design methods R_{pred} . Note that a value of $R_{\text{shell}}/R_{\text{pred}}$ greater than 1.0 indicates a safe sided resistance prediction by the considered design method and vice versa.

7.4.1 Members subjected to bending

To assess the accuracy of the proposed method for the design of HSS welded I-section members subjected to bending, a total of 400 simply supported welded I-section beams made of S355 and S690 steel with varying local cross-section slenderness have been analysed under three- and four-point bending. The normalised bending moment capacities M_u/M_{el} for all the S355 and S690 steel welded I-section beams subjected to three-point bending are plotted in Figures 7.11 and 7.12, respectively, in which M_{el} is the elastic moment capacities defined as the product of the elastic section modulus W_{el} and the yield stress f_y . Moving averages over 5 adjacent data points of the benchmark shell FE results and the beam FE results obtained using the proposed design method are also plotted in each figure. The resistances of Class 3 cross-sections can

alternatively be determined from elasto-plastic bending moment capacities M_{ep} , defined as the product of the elasto-plastic section modulus W_{ep} (linear interpolation between W_{el} and plastic section modulus W_{pl}) and the yield stress f_y . As shown in Figures 7.11 and 7.12, the proposed design method with an upper limit of $\Omega = 15$ for NSS and $\Omega = 10$ for HSS generally provides more accurate and consistent bending moment resistance predictions than the EC3 design rules. For NSS cross-sections with $\bar{\lambda}_p \leq 0.32$, a greater amount of material strain hardening can be exploited by adopting a higher upper bound, such as $\Omega = 30$ (Fieber et al., 2019b; Quan et al., 2020). For HSS cross-sections with $\bar{\lambda}_p \leq 0.36$, the proposed design method, with the upper limit of $\Omega = 15$ and $\Omega = 10$, also provides very accurate resistance predictions, while more safe side predictions were offered when using the value of 10; this value is therefore recommended for HSS design given the lower ductility. The proposed design method provides slightly conservative resistance predictions for HSS cross-sections with $\bar{\lambda}_p > 0.68$ (i.e. Class 4 cross-sections), mainly because the CSM base curve predicts the relationship between strain ratio and slenderness for slender cross-sections more conservatively than that for non-slender cross-sections (Gardner et al., 2023).

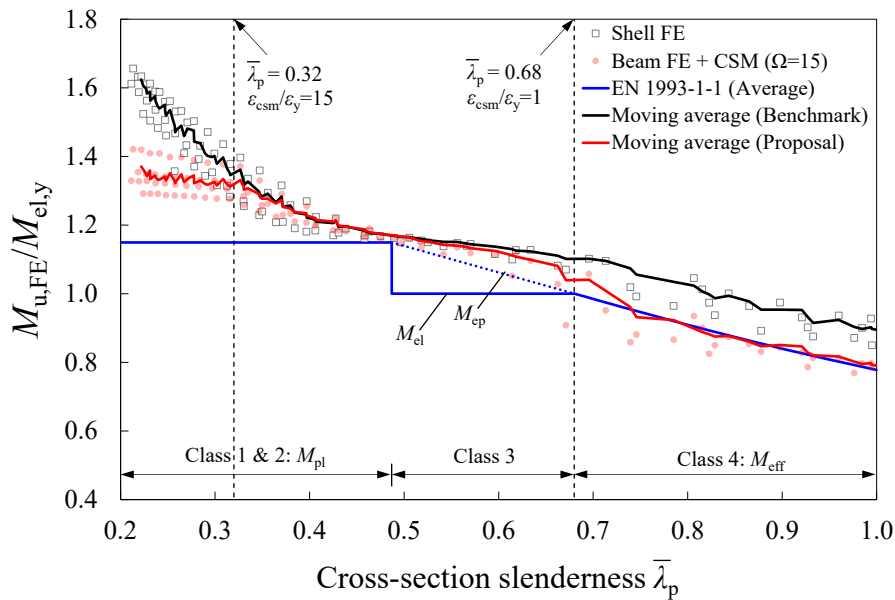


Figure 7.11: Bending resistance predictions for S355 steel welded I-sections of varying slenderness subjected to three-point bending (with moving averages over 5 adjacent data points).

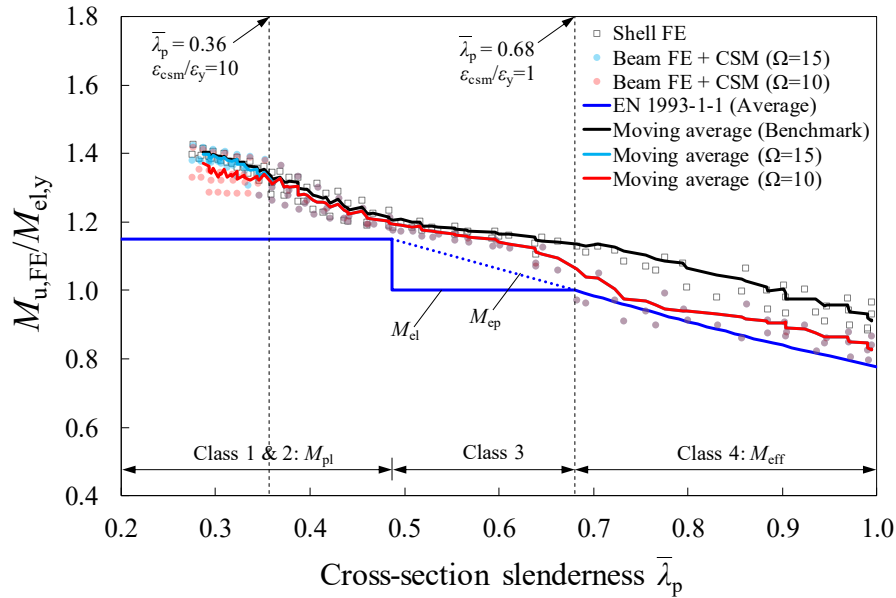


Figure 7.12: Bending resistance predictions for S690 steel welded I-sections of varying slenderness subjected to three-point bending (with moving averages over 5 adjacent data points).

A statistical analysis of the ratios of the benchmark shell FE results over the bending moment resistance predictions determined using the considered design methods (i.e. $M_{u,shell}/M_{pred}$) is summarised in Table 7.5. The current EC3 design rules appear to be conservative for both NSS and HSS, with the overall mean and COV values of $M_{u,shell}/M_{pred}$ equal to 1.087 and 0.075, respectively, whereas both the accuracy and the consistency of the resistance predictions are improved when using the proposed design method with an upper limit of $\Omega = 15$ for NSS and $\Omega = 10$ for HSS, with the overall mean and COV values of $M_{u,shell}/M_{pred}$ equal to 1.038 and 0.063, respectively.

Table 7.5: Comparisons of the benchmark shell FE results ($M_{u,shell}$) against the bending resistance predictions determined using the proposed design method and prEN 1993-1-1 (M_{pred}) for S355 and S690 steel welded I-section members subjected to three- and four-point bending.

Steel grade	No. of cases	Design method	$M_{u,shell}/M_{pred}$			
			Mean	COV	Maximum	Minimum
S690	100	EN 1993-1-1	1.088	0.073	1.281	0.961
		AA + CSM ($\Omega=15$)	1.022	0.062	1.200	0.822
		AA + CSM ($\Omega=10$)	1.033	0.054	1.200	0.936
S355	100	EN 1993-1-1	1.086	0.078	1.281	0.932
		AA + CSM ($\Omega=15$)	1.044	0.071	1.213	0.922

7.4.2 Members subjected to compression

To assess the accuracy of the proposed method for the design of HSS welded I-section members subjected to compression, a total of 426 pin-ended welded I-section columns made of grade S355 and S690 steel were analysed, considering major and minor axis buckling, three values of non-dimensional member slenderness ($\bar{\lambda} = 0.5, 1.1, 1.7$) and varying cross-section slenderness values $\bar{\lambda}_p$ ranging from 0.2 to 1.0. The benchmark shell FE results as well as the ultimate buckling capacities obtained using the proposed design method and the current EC3 design approach were normalised by the plastic squash load N_{pl} , which is a product of the gross cross-section area A and the yield stress f_y , and plotted against the cross-section slenderness $\bar{\lambda}_p$ in Figures 7.13-7.16.

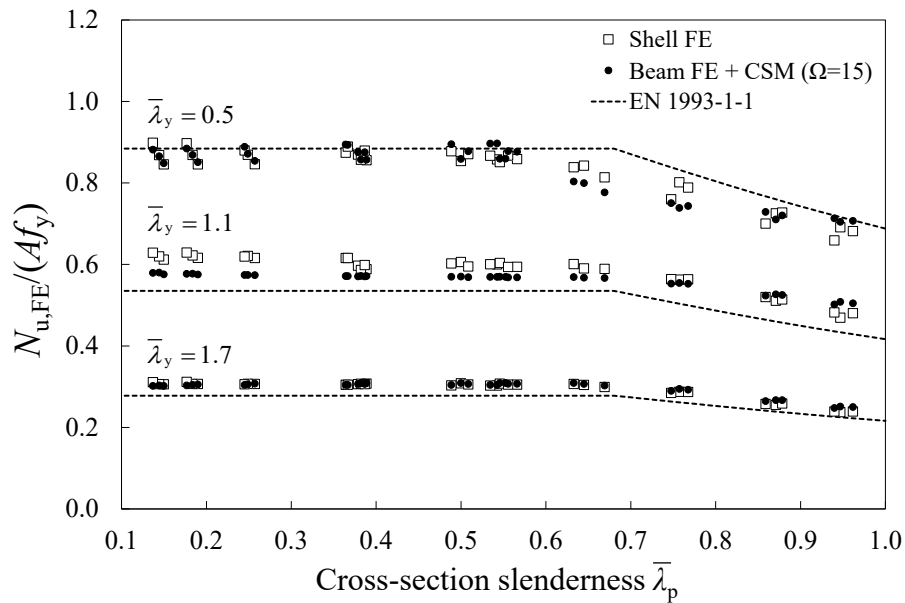


Figure 7.13: Resistance predictions of S355 steel welded I-section columns of varying cross-section slenderness $\bar{\lambda}_p$ buckling about the major axis for three values of member slenderness ($\bar{\lambda} = 0.5, 1.1, 1.7$).

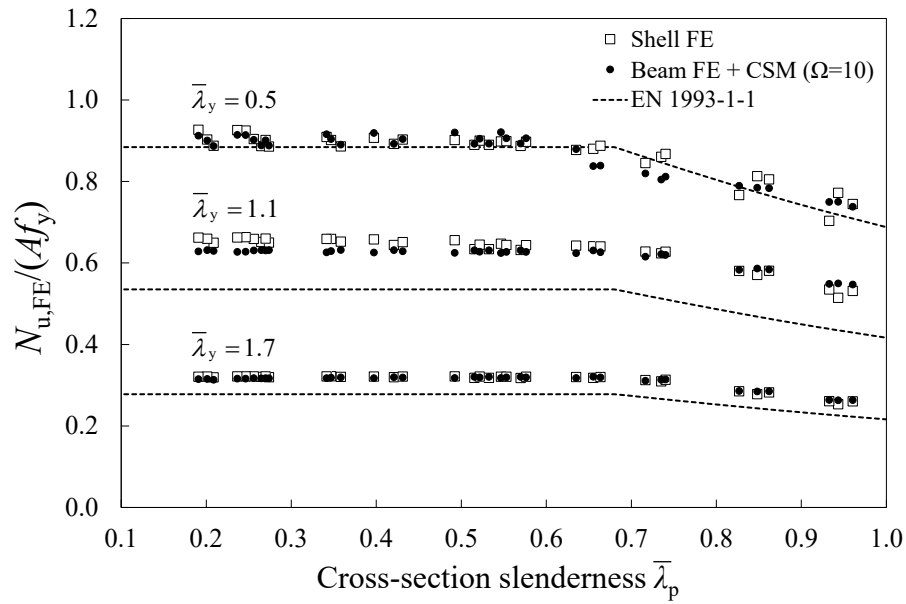


Figure 7.14: Resistance predictions of S690 steel welded I-section columns of varying cross-section slenderness $\bar{\lambda}_p$ buckling about the major axis for three values of member slenderness ($\bar{\lambda} = 0.5, 1.1, 1.7$).

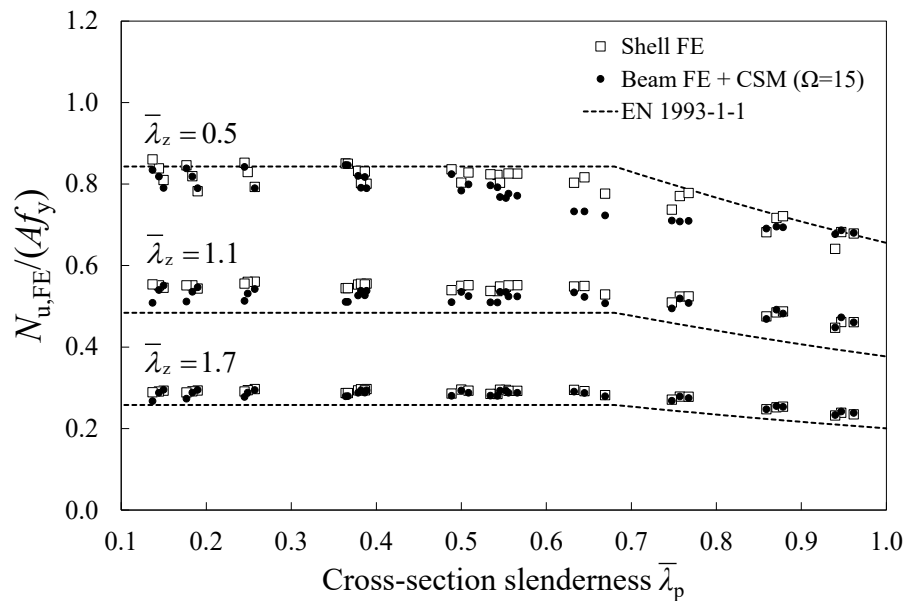


Figure 7.15: Resistance predictions of S355 steel welded I-section columns of varying cross-section slenderness $\bar{\lambda}_p$ buckling about the minor axis for three values of member slenderness ($\bar{\lambda} = 0.5, 1.1, 1.7$).

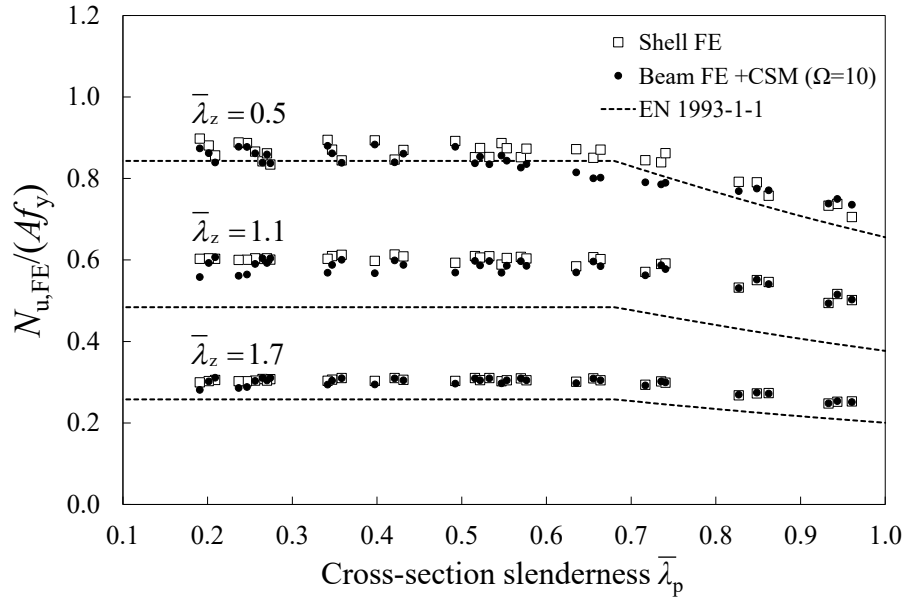


Figure 7.16: Resistance predictions of S690 steel welded I-section columns of varying cross-section slenderness $\bar{\lambda}_p$ buckling about the minor axis for three values of member slenderness ($\bar{\lambda} = 0.5, 1.1, 1.7$).

A statistical analysis of the ratios of $N_{u,shell}/N_{pred}$ is reported in Table 7.6, in which $N_{u,shell}$ is the benchmark shell FE ultimate capacity and N_{pred} is the ultimate resistance predicted using the considered design method. Note that for columns with slender cross-sections (e.g. $\bar{\lambda}_p > 0.6$), the resistance predictions obtained from the proposed design method were mostly governed by the attainment of the CSM strain limit, while for columns with less slender cross-sections (e.g. $\bar{\lambda}_p < 0.6$), the resistance predictions were generally governed by the peak load factors. Some of the predictions for short columns with very slender cross-sections fall on the unsafe side, owing to the neglect of the effects of local-global interactive buckling in the proposed design method. It is found that the proposed design method generally provides more accurate and less scattered resistance predictions relative to the current EC3 design approach, with the overall mean and COV values of $N_{u,shell}/N_{pred}$ respectively equal to 1.016 and 0.030.

Table 7.6: Comparisons of the benchmark shell FE results $N_{u,shell}$ against the ultimate buckling resistance N_{pred} predicted using the proposed design method and prEN 1993-1-1 for welded I-section columns made of S355 and S690 steel.

Buckling axis	Steel grade	Design method	$N_{u,shell}/N_{pred}$			
			Mean	COV	Max	Min
Major	S690	EN 1993-1-1	1.126	0.075	1.240	0.972
		AA+CSM ($\Omega=10$)	1.010	0.024	1.069	0.936
	S355	EN 1993-1-1	1.064	0.068	1.176	0.907
		AA+CSM ($\Omega=15$)	1.011	0.037	1.090	0.923
	Both	EN 1993-1-1	1.094	0.077	1.240	0.907
		AA+CSM	1.010	0.031	1.090	0.923
Minor	S690	EN 1993-1-1	1.157	0.080	1.315	0.989
		AA+CSM ($\Omega=10$)	1.018	0.024	1.092	0.959
	S355	EN 1993-1-1	1.081	0.074	1.184	0.923
		AA+CSM ($\Omega=15$)	1.026	0.028	1.114	0.946
	Both	EN 1993-1-1	1.118	0.084	1.315	0.923
		AA+CSM	1.022	0.027	1.114	0.946
Overall		EN 1993-1-1	1.106	0.081	1.315	0.907
		AA+CSM	1.016	0.030	1.114	0.923

7.4.3 Members subjected to combined compression and uniaxial bending

In this section, the accuracy of the proposed method for the in-plane stability design of HSS welded I-section members is examined against 3960 pin-ended beam-columns subjected to combined compression and uniaxial bending. Both major and minor axis buckling were considered, together with three values of member slenderness ($\bar{\lambda} = 0.5, 1.1, 1.7$) and three bending moment distributions along the member length ($\psi = 1.0, 0, -0.5$). As illustrated in Table 7.3, different bending moment distributions ψ were achieved by changing the ratio of the applied end moments (i.e. M_2/M_1), where $\psi = 1.0$ refers to uniformly distributed bending moment, $\psi = 0$ refers to linearly distributed bending moment with $M_2 = 0$, and $\psi = -0.5$ refers to linearly distributed bending moment with $M_2 = -0.5M_1$. Twelve different cross-section sizes of varying local slenderness were considered for each steel grade. The ratio of applied compression to bending was varied to cover the full range of loading scenarios from compression alone to bending alone.

Normalised moment-compression (M - N) interaction relationships for a series of beam-column examples made of S690 steel subjected to combined compression and uniaxial bending with $\psi = 1.0$, $\psi = 0$, and $\psi = -0.5$ are illustrated in Figures 7.17-7.19, respectively. In the figures, the benchmark shell FE results are compared against the ultimate capacities predicted using three different design methods, including (1) the EC3 member buckling checks, (2) linear interaction cross-section checks on results of second order elastic analyses (i.e. beam element GNIA + EC3 cross-section checks), and (3) the proposed design method (i.e. beam element GMNIA + CSM strain limits). When predicting the resistances of members with Class 3 cross-sections using the two EC3 design methods, the elasto-plastic section moduli were employed, as set out in prEN 1993-1-1 (2019). The performed GNIA incorporated the elastic reference equivalent bow imperfections set out in prEN 1993-1-1 (2019), which are equal to $L/110$ for members buckling about the major axis and equal to $L/200$ for members buckling about the minor axis, where L is the member length. It can be seen from Figures 7.17-7.19 that the two EC3 design methods result in slightly more conservative resistance predictions for members subjected to compression plus uniform bending (i.e. $\psi = 1$), and somewhat inaccurate, though generally safe sided, resistance predictions for members subjected to compression plus non-uniform bending (i.e. $\psi = 0$ and $\psi = -0.5$). On the other hand, the proposed design method provides generally safe sided resistance predictions that agree well with the benchmark shell FE results. According to the proposed design method, for the short members and for the bending dominated cases, the resistance predictions are determined by reaching the CSM strain limits, while for the longer members and for the compression dominated cases, the resistance predictions correspond to the peak load factors during the analyses.

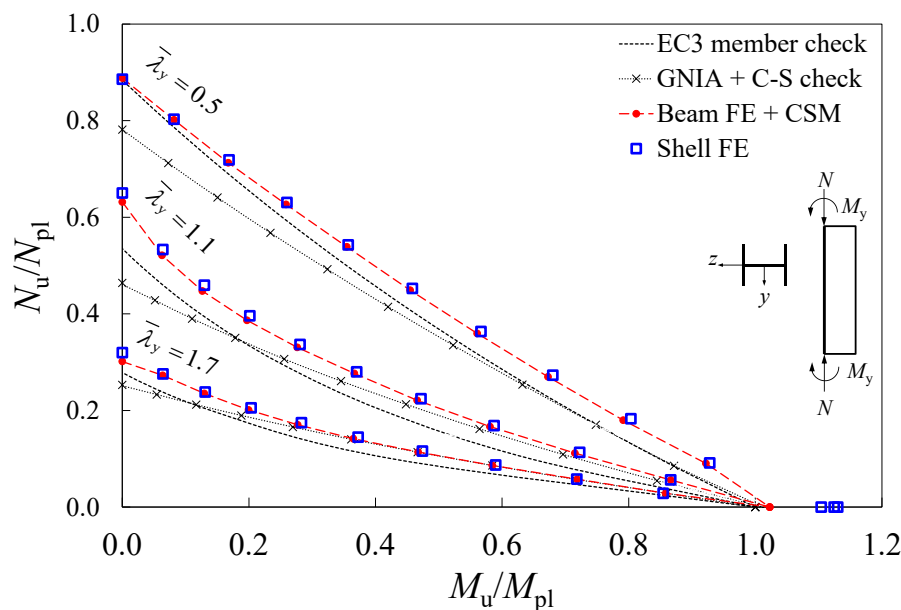


Figure 7.17: Comparison of the benchmark shell FE results against the normalised ultimate capacities predicted using different design methods for S690 steel pin-ended beam-columns with different member slenderness and $\psi = 1.0$ under combined compression plus major axis bending (Class 1 welded I-section: $100 \times 100 \times 16 \times 7.8$ in mm, $\bar{\lambda}_p = 0.27$ in compression).

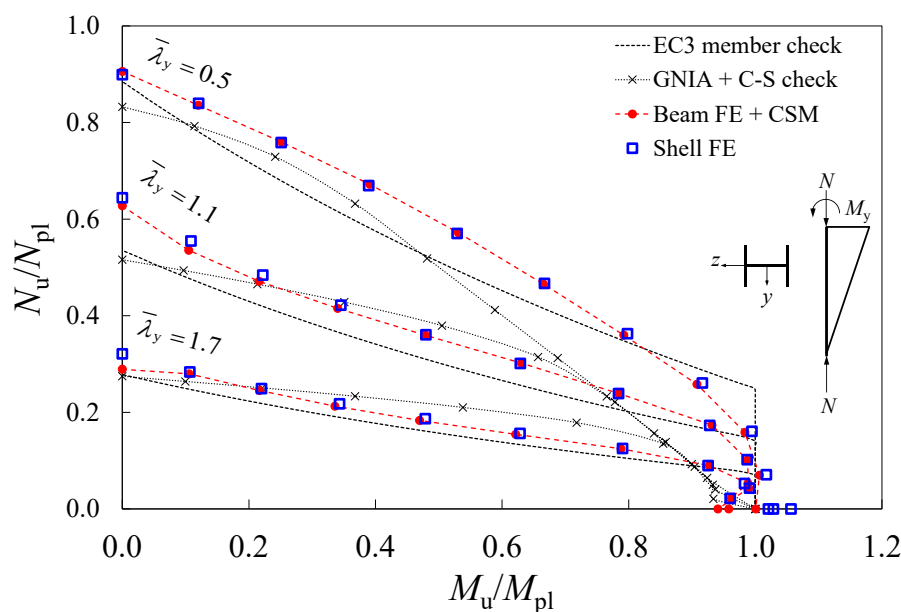


Figure 7.18: Comparison of the benchmark shell FE results against the normalised ultimate capacities predicted using different design methods for S690 steel pin-ended beam-columns with different member slenderness and $\psi = 0$ under combined compression plus major axis bending (Class 3 welded I-section: $100 \times 150 \times 7.8 \times 7.5$ in mm, $\bar{\lambda}_p = 0.55$ in compression).

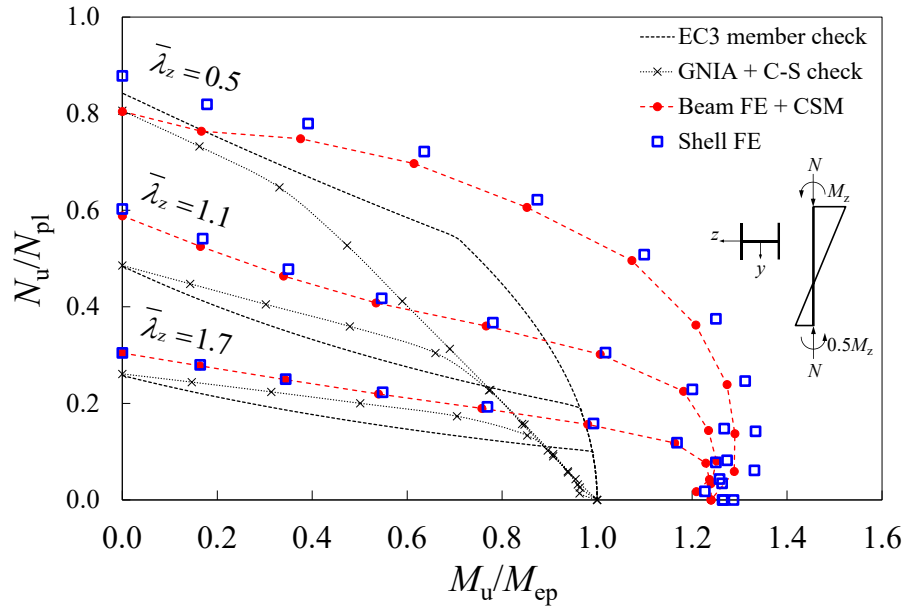


Figure 7.19: Comparison of the benchmark shell FE results against the normalised ultimate capacities predicted using different design methods for S690 steel pin-ended beam-columns with different member slenderness and $\psi = -0.5$ under combined compression plus minor axis bending (Class 3 welded I-section: $100 \times 150 \times 5.8 \times 5.7$ in mm, $\bar{\lambda}_p = 0.74$ in compression).

Similar conclusions can be drawn from Figure 7.20 and Table 7.7. Figure 7.20 compares the radial distances measured from the origin to the datapoint of the benchmark shell FE results (R_{shell}) against those measured from the origin to the datapoint determined using the considered design method (R_{pred}) for all the considered cases. Statistical analyses on the ratios of $R_{\text{shell}}/R_{\text{pred}}$ are reported in Table 7.7. The ratios of $R_{\text{shell}}/R_{\text{pred}}$ are plotted against the corresponding radial angle parameters θ in Figure 7.20, which are defined in Figure 7.10 to describe the relationship between the axial compressive load and the bending moment. The angle parameter θ varies between 0° and $\theta = 90^\circ$, for which $\theta = 0^\circ$ and $\theta = 90^\circ$ correspond to bending alone and compression alone, respectively.

Table 7.7: Comparisons of the benchmark shell FE results R_{shell} against the ultimate capacities R_{pred} predicted using different design methods for welded I-section beam-columns made of S355 and S690 steel.

Buckling axis	Moment ratio ψ	No. of cases	Steel grade	$R_{\text{shell}}/R_{\text{pred}}$											
				AA+CSM				EC3 member check				GNIA+EC3 C-S check			
				Mean	COV	Max	Min	Mean	COV	Max	Min	Mean	COV	Max	Min
Major axis	1	396	S690	1.018	0.033	1.152	0.969	1.067	0.044	1.238	0.979	1.086	0.067	1.453	0.962
		396	S355	1.015	0.038	1.240	0.939	1.024	0.041	1.157	0.917	1.066	0.065	1.341	0.947
		792	Both	1.016	0.036	1.240	0.939	1.045	0.047	1.238	0.917	1.076	0.067	1.453	0.947
	0	396	S690	1.031	0.049	1.254	0.964	1.109	0.065	1.369	0.976	1.079	0.091	1.453	0.866
		396	S355	1.022	0.053	1.333	0.937	1.069	0.061	1.305	0.913	1.039	0.093	1.341	0.825
		792	Both	1.026	0.051	1.333	0.937	1.089	0.066	1.369	0.913	1.059	0.094	1.453	0.825
	-0.5	396	S690	1.036	0.050	1.231	0.957	1.193	0.156	1.976	0.931	1.131	0.070	1.453	0.994
		396	S355	1.025	0.058	1.305	0.948	1.151	0.149	1.927	0.911	1.089	0.062	1.341	0.968
		792	Both	1.030	0.054	1.305	0.948	1.172	0.154	1.976	0.911	1.110	0.069	1.453	0.968
	Overall	1188	S690	1.028	0.045	1.254	0.957	1.123	0.115	1.976	0.931	1.099	0.079	1.453	0.866
		1188	S355	1.020	0.051	1.333	0.937	1.082	0.112	1.927	0.911	1.065	0.076	1.341	0.825
		2376	Both	1.024	0.048	1.333	0.937	1.102	0.115	1.976	0.911	1.082	0.080	1.453	0.825
Minor axis	1	396	S690	1.033	0.054	1.431	0.928	1.120	0.088	1.509	0.902	1.169	0.143	1.762	0.880
		396	S355	1.029	0.041	1.295	0.914	1.118	0.120	1.542	0.891	1.158	0.147	1.630	0.872
		792	Both	1.031	0.048	1.431	0.914	1.119	0.105	1.542	0.891	1.164	0.145	1.762	0.872
	-0.5	396	S690	1.058	0.074	1.576	0.940	1.189	0.145	2.065	0.924	1.240	0.136	1.767	0.944
		396	S355	1.044	0.055	1.382	0.910	1.195	0.197	2.201	0.923	1.244	0.157	1.869	0.937
		792	Both	1.051	0.066	1.576	0.910	1.192	0.173	2.201	0.923	1.242	0.147	1.869	0.937
	Overall	792	S690	1.045	0.066	1.576	0.928	1.155	0.125	2.065	0.902	1.205	0.142	1.767	0.880
		792	S355	1.037	0.049	1.382	0.910	1.157	0.168	2.201	0.891	1.201	0.157	1.869	0.872
		1584	Both	1.041	0.059	1.576	0.910	1.156	0.148	2.201	0.891	1.203	0.150	1.869	0.872
	Overall	3960	Both	1.031	0.053	1.576	0.910	1.124	0.132	2.201	0.891	1.130	0.128	1.869	0.825

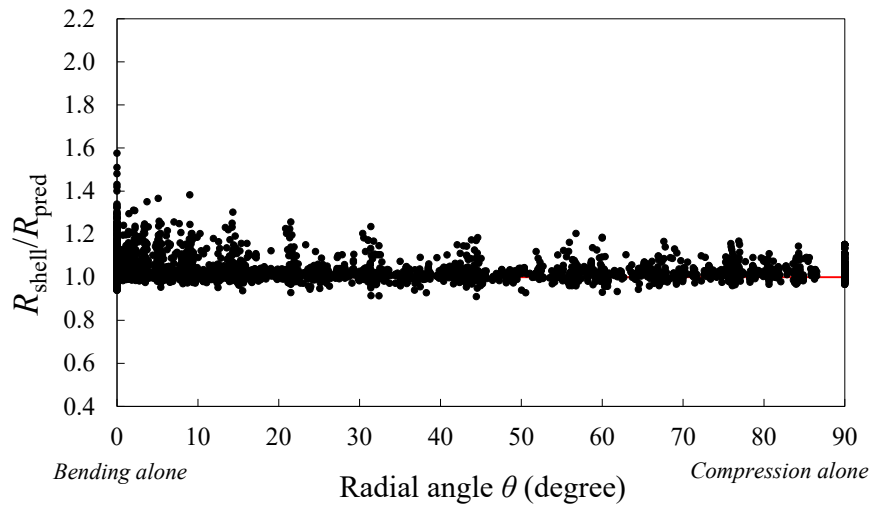
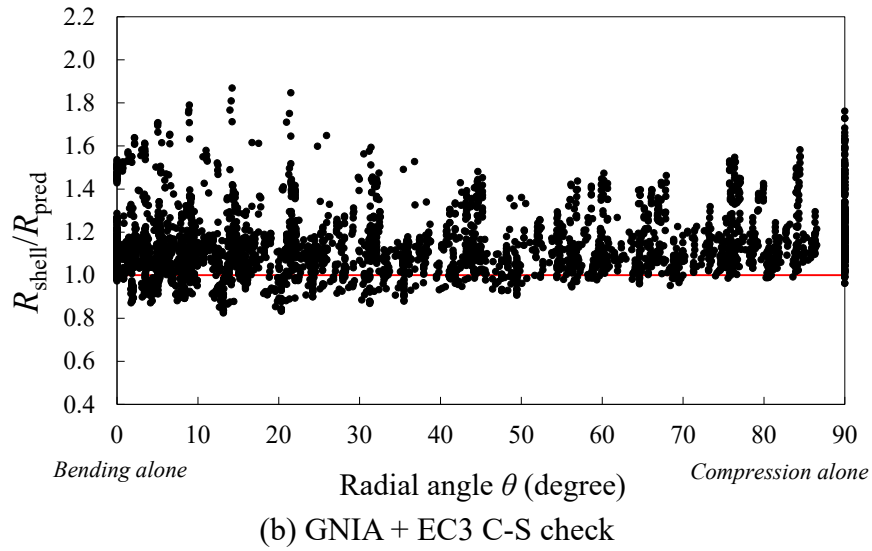
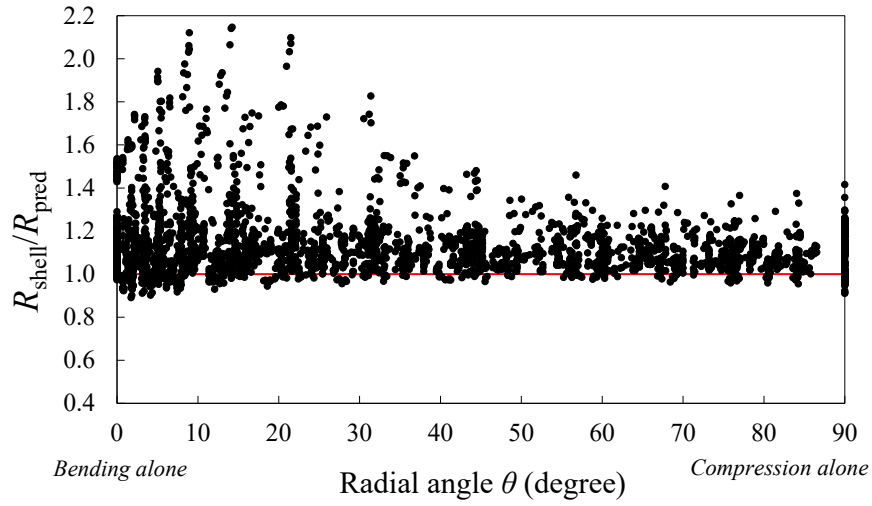


Figure 7.20: Comparisons of the benchmark shell FE results against the resistance predictions obtained using different design methods for all the considered cases (with respect to radial angle parameter θ).

Overall, the proposed design method yields the most accurate but still generally safe sided resistance predictions for both NSS and HSS welded I-section beam-columns, with the overall mean and COV values of $R_{\text{shell}}/R_{\text{pred}}$ equal to 1.031 and 0.053, respectively. The resistance predictions offered by the two EC3 design approaches are rather scattered and conservative, particularly for bending dominated cases. The overall mean and COV values of $R_{\text{shell}}/R_{\text{pred}}$ are equal to 1.124 and 0.132, respectively, when performing the EC3 member buckling checks, while they are equal to 1.130 and 0.128, respectively, when applying linear interaction cross-section checks using GNIA. The significant benefits of the proposed design method over the current EC3 design approaches result from the following aspects: (1) the interaction between flange and web plates are accounted for in the determination of cross-section local slenderness $\bar{\lambda}_p$, (2) the beneficial effects of material strain hardening and local moment gradients in the determination of ultimate capacities are rationally exploited through the CSM and the strain averaging approach, (3) the second order effects and spread of plasticity are directly captured in the GMNIA analysis, and (4) the need for separate member checks (including the calculation of effective lengths and moment gradient factors) as well as the calculation of effective section moduli W_{eff} for Class 4 cross-sections are eliminated.

7.5 Conclusions

In this chapter, a recently proposed advanced design framework that employs computationally efficient beam element models to perform GMNIA and utilises strain limits, defined by the Continuous Strength Method (CSM), to simulate local buckling behaviour is presented and extended to both NSS and HSS welded I-section structural members subjected to different loading scenarios. Shell FE models were created and validated against existing test data on welded I-section structural members, and subsequently used in a comprehensive parametric study to generate benchmark FE results for the assessment of the proposed design method, considering a broad range of cross-section slendernesses, member slendernesses and different loading conditions. The assessment revealed that the proposed design method, with an upper CSM strain ratio limit of $\Omega = 15$ for NSS and $\Omega = 10$ for HSS, results in consistently more accurate resistance predictions relative to the current EC3 design method set out in EN 1993-1-1 (2005, 2019). Further research is currently underway to extend this practical advanced design approach to structural steel indeterminate structures.

Chapter 8

Conclusions and further research

8.1 Conclusions

A comprehensive study into the behaviour and design of high strength steel structural elements and frames has been presented in this thesis. This chapter briefly summarises the key findings and overall conclusions of this study. More detailed concluding remarks have been provided at the end of each chapter. Suggestions for further research are also provided in Section 8.2.

The development, advantages and application of high strength steels in engineering practice over the past decades was firstly introduced. A general review of previous research into the structural performance and design of determinate and indeterminate structures made of high strength steel was then presented, from which test data were collected for use later in the validation of numerical models and the evaluation of different design methods. However, it was found that high quality test data on high strength steel, particularly for indeterminate structures, remain limited. This has restricted the extension of plastic global analysis to high strength steel, setting a barrier to more efficient structural design and more cost-effective design solutions.

To address the aforementioned gaps in available test data, an experimental programme was firstly carried out in the Structures Laboratory at Imperial College London to investigate the in-plane structural behaviour of members and frames made of S690 welded I-sections. The experimental programme consisted of four tensile coupon tests, five major axis flexural buckling tests on long columns, six three-point bending tests and eight tests on single storey,

single bay unbraced rectangular steel frames. Three welded I-section profiles were considered, all Class 1 according to the slenderness limits set out in the latest version of the European design standard prEN 1993-1-1:2019 (2019). Cross-section geometries and initial geometric imperfections for each test specimen were measured prior to testing. Apart from traditional measuring instrumentation, digital image correlation (DIC) was also employed in the three-point bending tests and the frame tests in order to provide more comprehensive displacement and strain measurements over the entire monitored surface.

In parallel with the experimental programme, numerical parametric studies were conducted to create supplementary and more extensive structural performance data for HSS structures with varying welded I-sections. Finite element (FE) models using shell elements were created and employed a novel residual stress model that was proposed based on a large amount of existing residual stress measurements for welded I-sections made of S235 to S960 steel. The validity of the FE models was confirmed by comparing against key test results and failure modes obtained from the experimental programme, as well as those collected from the literature. Extensive numerical parametric studies were then carried out using the validated FE models, through which more than 15000 numerical data on NSS and HSS welded I-section members were generated, covering a wide range of steel grades, cross-section geometries, member slendernesses and loading scenarios. The generated numerical data, together with the collected test results, were used to assess the accuracy of the current design approaches (as set out in the European (EC3) and North American (AISC) design standards), as well as new design proposals, for NSS and HSS welded I-section members subjected to different loading conditions in the relevant chapters.

At the cross-section level, the ultimate bending moment capacities for welded I-sections made of different steel grades generally follow the same trend. An underestimation of bending moment capacities was found for very stocky cross-sections, due primarily to the beneficial effects of material strain hardening being neglected in the current cross-section design framework; this is more prominent for NSS cross-sections with lower yield strengths. The suitability of applying the current EC3 cross-section slenderness limits to HSS homogeneous and hybrid welded I-sections in bending was also evaluated. It was shown that the current EC3 Class 2 and Class 3 slenderness limits were suitable for the classification of the outstand flange (in compression) and internal web (in bending) elements of both HSS homogeneous and hybrid welded I-sections subjected to major axis bending, while stricter Class 1 slenderness limits

were considered necessary to achieve sufficient rotation capacity for plastic design. The results indicated that plastic design may be applied to HSS structures, provided the proposed stricter Class 1 slenderness limits are employed. The potential of extending plastic design to HSS indeterminate structures was then demonstrated by carrying out experimental studies on HSS frames in Chapter 6. The experimental investigations revealed that, despite the lower ductility and strain hardening of HSS relative to NSS, HSS frames with stocky cross-sections have the ability to form plastic hinges with sufficient rotation capacity to achieve a considerable amount of inelastic moment redistribution, and allow the application of plastic design.

At the member buckling level, the accuracy of the flexural buckling design rules for NSS and HSS welded I-section columns set out in the current European (EC3) and North American (AISC) design standards were firstly evaluated, where shortcomings relating to the consideration of steel grade were identified. Both the EC3 and AISC design approaches yield flexural buckling resistance predictions with varying levels of accuracy for columns made of different steel grades, which is attributed to the fact that only a single column buckling curve is used for the design of welded I-section columns buckling about each axis, regardless of the steel grade. To address this issue, a new design method was devised within the EC3 column design framework to reflect the influence of yield strength on the buckling resistances of welded I-section columns more systematically and shown to provide improved resistance predictions in terms of accuracy and consistency.

For members under combined loading, as discussed in Chapter 5, owing to the inaccuracy of the interaction curve end points and the interaction curves themselves, the current EC3 and AISC design methods yield varying levels of accuracy in predicting the ultimate capacities of welded I-section beam-columns depending on the steel grade. New design proposals that are compatible with the EC3 beam-column design framework were developed, featuring more accurate yield strength-dependent interaction curves anchored to improved resistance predictions for members subjected to isolated loading scenarios (i.e. the compression and bending end points of the interaction curves). Specifically, the compression end points of the modified interaction curves are determined using the modified EC3 design method proposed in Chapter 3, while the bending end points of the modified interaction curves are calculated using the bending moment resistance functions defined by the continuous strength method (CSM).

The current EC3 and AISC design standards for steel structures use the traditional two-step design framework (i.e. structural analysis plus individual member checks) for ease of hand calculation in engineering practice; however, the traditional design approaches are typically carried out based on first order elastic structural analysis and cannot directly capture second order effects and the spread of material nonlinearity within structures. The prohibition of plastic design for HSS structures was also questioned in the present study and some recent research (Bartsch et al., 2021b; Gkantou et al., 2018). To overcome these issues, a practical advanced analysis-based design method that incorporates the continuous strength method (CSM) was devised by Fieber et al. (2019a) for the design of NSS members. In this design method, geometrically and materially nonlinear analyses with imperfections (GMNIA) are firstly performed using beam finite elements, with the ultimate capacities being determined according to either the peak load factors or the load factors at which CSM strain limits are attained, whichever occurs first. The CSM strain limits are used to allow for the effects of cross-section failure as well as define a rational allowance of material nonlinearity (i.e. the spread of plasticity and material strain hardening) in the determination of ultimate strength. The beneficial effects of local moment gradients can also be exploited by applying the strain limit to compressive strains averaged over a defined distance along the member length. In Chapter 7, the scope of this design method was successfully extended to the in-plane design of HSS welded I-section members. Comparisons against the current EC3 design approaches revealed that the proposed design method can provide consistently more accurate and still safe sided resistance predictions for HSS welded I-section members.

8.2 Suggestions for further research

On the basis of the present study, further research could be conducted in the following aspects. The present research studied the global buckling behaviour of NSS and HSS members with non-slender (Class 1-3) welded I-sections under different loading conditions and devised new design proposals compatible with the EC3 design framework with consideration of steel grades. Further research is needed to extend the scope of the new design proposals to NSS and HSS members with slender (Class 4) welded I-sections, the responses of which are affected by local-global buckling interaction (Bai & Wadee, 2016; Liu & Wadee, 2016; Shen & Wadee, 2019a, 2019b; Shi et al., 2020). It is also required to evaluate the accuracy of applying the new design

proposals to NSS and HSS welded I-section beam-columns subjected to compression plus biaxial bending. Welded I-section members may be more susceptible to out-of-plane failure if not fully laterally restrained, such as lateral torsional buckling and torsional buckling, which is also worth studying in the future.

The experimental studies on HSS frames at the structural system level presented in this thesis have allowed the assessment of the interaction between plastic collapse and global instability, and provide benchmark results that are suitable for the validation of finite element models and the development of system-level design approaches in further research. In the future, numerical investigations could be conducted to examine the susceptibility of HSS structures to second order effects, as well as to evaluate further the applicability of the conventional global plastic design rules, as set out in the EN 1993-1-1 (2005, 2019), to HSS frames.

Considering the rapid developments in computational power and software, it is possible to promote the scope of the practical advanced analysis-based design method into engineering practice. The present study successfully extended this design method to the in-plane design of HSS welded I-section members. In order to make this design method a complete and practical design tool and to encourage its wider use in structural applications with HSS, extensions of the design method to cover out-of-plane failure of HSS members and HSS indeterminate structures, including continuous beams, planar steel frames and three-dimensional steel frames, could be carried in the future.

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