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FLEXURAL BUCKLING OF HOT-FINISHED HIGH STRENGTH STEEL SHS AND RHS COLUMNS

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ABSTRACT

1 An experimental and numerical study of the flexural buckling behavior of hot-finished
2 high strength steel (HSS) square and rectangular hollow section (SHS and RHS) columns is
3 described in this paper. A total of 30 hot-finished S460 and S690 hollow section column spec-
4 imens have been tested in compression with pin-ended boundary conditions. Finite element
5 (FE) models have been developed to replicate the experiments, and employed in a subsequent
6 parametric study considering a range of member geometries. Based on the test and FE results,
7 the applicability of the current column design curves in European, North American, Chinese
8 and Australian structural steel design standards to hot-finished HSS SHS and RHS columns
9 has been verified by means of reliability analyses.

10 **Keywords:** AISC 360, AS 4100, Buckling, Column, Eurocode 3, Experiment, GB
11 50017, High strength steels, Hot-finished hollow sections, Member instability, Testing,
12 SHS, RHS, Reliability analysis

13 INTRODUCTION

14 The development of modern production techniques, such as thermo-mechanical
15 rolling and quenching and tempering, has enabled high strength steels (HSS) with

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16 yield strengths up to 1100 N/mm^2 , while retaining good weldability and adequate
17 toughness and ductility, to be produced (IABSE 2005). Their potential in structural
18 applications has been demonstrated in existing landmark bridges and buildings, such
19 as the Millau Viaduct in France, the Australia Star City in Australia (Pocock 2006) and
20 the NRG Stadium in Houston, USA (Griffis et al. 2003). However, at present, there
21 is limited available structural design guidance. The American structural steel design
22 standard AISC 360-10 (AISC 2010) covers steel grades up to S690 (ASTM A514).
23 The Chinese code GB 50017 (MOHURD 2003) provides design rules for steel grades
24 up to S420. In Australia, the standard AS 4100 (Standards Australia 1998) originally
25 covered steels with yield strengths up to 450 N/mm^2 , but a recent amendment AS
26 4100 AMDT 1-2012 (Standards Australia 2012) extended the scope of application up
27 to 690 N/mm^2 . In each of these codes, high strength steels are essentially treated in the
28 same way as conventional strength steels. In Europe, EN 1993-1-12 (CEN 2007) was
29 published specifically for the structural design of high strength steels (S460-S700), but
30 is again, essentially, a simple extension of the conventional carbon steel design rules
31 provided in EN 1993-1-1 (CEN 2005).

32 The aim of the current study is to contribute towards the development of accurate
33 design rules for hot-finished HSS columns. Previous work on HSS columns include
34 experimental studies by Rasmussen and Hancock (1995) on S690 welded box- and I-
35 section columns and a number of more recent studies on welded HSS columns in steel
36 grades S460 (Ban et al. 2012; Wang et al. 2012, 2014), S690 (Shi et al. 2012), and S960
37 (Shi et al. 2012; Ban et al. 2013). In these studies, it was generally concluded that HSS
38 columns possess higher normalized buckling resistances than their conventional carbon
39 steel counterparts, which can be attributed to the reduced sensitivity of HSS members
40 to geometric imperfections and lower residual stresses in HSS sections as a proportion
41 of the yield strength (IABSE 2005).

42 Since previous studies have mainly focused on welded sections, this paper de-

43 scribes a series of experiments on hot-finished S460 and S690 tubular members. Rig-
44 orous finite element (FE) models are developed and validated against the test results,
45 and subsequently employed to generate parametric results. Based on the test and FE
46 results, statistical reliability analyses are performed to assess the applicability of the
47 EC 3 (CEN 2005; 2007) HSS column design curve, and also those in North American
48 (AISC 2010), Chinese (MOHURD 2003) and Australian (Standards Australia 1998)
49 conventional carbon steel design standard, on hot-finished HSS tubular members.

50 **EXPERIMENTAL STUDY**

51 A comprehensive testing programme on hot-finished S460NH and S690QH square
52 and rectangular hollow section (SHS and RHS) elements has been carried out. The
53 experiments were designed to cover different structural aspects at material level (Wang
54 et al. 2017), cross-section level (Wang et al. 2016, 2017; Gkantou et al. 2016) and
55 member level. Focusing on the member behavior, this paper describes a series of test-
56 s on SHS columns; RHS columns are studied numerically, as detailed later in this
57 paper. The test specimens comprised five cross-section sizes: S460 SHS $50\times 50\times 5$,
58 $70\times 70\times 6.3$ and $100\times 100\times 5$, and S690 SHS $50\times 50\times 5$ and $100\times 100\times 5.6$. Although
59 these section sizes are at the smaller end of the commercially available range, the
60 proportions of the test specimens, which is the dominant factor in controlling their
61 buckling response, were representative of typical compression members found in prac-
62 tice and no significant influence from size effects would be anticipated. The S460NH
63 and S690QH sections were hot-rolled from continuously cast round ingots which were
64 then hollowed out in a piercing mill to the final section shape. The S460NH sec-
65 tions were subsequently normalised, while the S690QH sections were quenched and
66 tempered. For both materials, the resulting sections are categorised as hot-finished
67 seamless tubes. Tensile material coupon tests were performed on all material.

68 **Tensile Coupon Tests**

69 To determine the engineering stress-strain response of the material, tensile coupon-
70 s extracted from both the flat and corner regions of each cross-section were tested in
71 accordance with EN ISO 6892-1 (CEN 2009). The detailed testing procedure has been
72 described by Wang et al. (2017), while a summary of the flat (F) and corner (C) coupon
73 test results is given herein in Table 2, where E is the material Young's modulus, f_y is
74 the (upper) yield strength, f_u is the ultimate tensile strength, and ε_f is the plastic s-
75 train at fracture, calculated based on the elongation over the standard gauge length
76 $5.65\sqrt{A_c}$, A_c being the cross-sectional area of the coupon (CEN 2009). Typical mea-
77 sured stress-strain curves are plotted in Fig. 1, showing the S460 SHS $50\times 50\times 5$ and
78 S690 SHS $50\times 50\times 5$ coupon results. Both steel grades display the anticipated sharply
79 defined yield point, yield plateau and subsequent strain hardening, as expected for hot-
80 finished materials, while the S690 materials present less strain hardening (indicated by
81 lower f_u/f_y ratios) and lower ductility (ε_f) than the S460 coupons, as can be seen in
82 Table 2 and Fig. 1. Given the observed similarity between the flat and corner mate-
83 rial, representative mean values of the flat coupon results, as summarized in Table 1,
84 were adopted for each cross-section in the subsequent member test result analysis and
85 numerical modeling.

86 **Residual stress measurements**

87 The existence of residual stresses in structural elements can cause premature yield-
88 ing, loss of stiffness and hence a reduction in load carrying capacity. Knowledge of
89 the residual stress distribution within steel cross-sections is therefore crucial. As part
90 of the present study, measurements of residual stresses in a hot-finished S690 SHS
91 $90\times 90\times 5.6$ specimen were made using the sectioning method, as detailed in Wang
92 et al. (2016). It was observed that no significant through-thickness bending residu-
93 al stresses existed, and that the axial membrane residual stresses were also low, with

94 maximum magnitudes of $0.055f_y$ in tension and $0.031f_y$ in compression, as shown in
95 Fig. 2. Based on the obtained results, a residual stress distribution model for hot-
96 finished high strength steel SHS and RHS was developed, as given in Figure 3. This
97 residual stress pattern was introduced into the numerical models for the FE validation
98 study, as described later.

99 **Flexural buckling tests**

100 To investigate the flexural buckling response of HSS tubular members, a total of
101 30 column specimens were tested. Five cross-section sizes with varying lengths en-
102 abled a wide spectrum of member slenderness (0.35-2.22) to be studied. A list of the
103 specimens, together with their designations and measured geometric dimensions, is
104 provided in Table 3, where h , b , t and r_i are the depth, width, wall thickness and inner
105 radius of the corner region of the cross-sections, respectively, as indicated in Fig. 6,
106 L_{cr} is the effective buckling length of the specimens, ω_i is the global imperfection am-
107 plitude derived from strain gauge readings following the procedure described later in
108 this section, and $\bar{\lambda}$ is the relative member slenderness as defined in EN 1993-1-1 (CEN
109 2005).

110 All the tests were performed using an Intron 2000 kN machine; the set-up is illus-
111 trated in Figs. 4 and 5 and is similar to that employed by Afshan and Gardner (2013)
112 and Chan and Gardner (2009). Pin-ended boundary conditions were achieved through
113 the use of hardened steel knife edge supports, allowing only in-plane rotation about
114 one-axis (i.e. the axis of buckling, as shown in Fig. 6). The distance between the top
115 and bottom knife edges was taken as the specimen buckling length, L_{cr} , which was
116 equal to the member cut-length plus 75 mm at each end, as shown in Fig. 4. The
117 specimens were loaded under displacement control at a loading rate of $L_{cr}/2000$ per
118 min. The monitored variables during the test included the applied axial load, the end-
119 shortening, the longitudinal strains and the lateral deflection at the mid-height of the

120 member. The axial load and displacement were obtained from the loading rig direct-
 121 ly. The longitudinal strains were measured by four linear electrical resistance strain
 122 gauges fixed at the mid-length of the member in the arrangement shown in Fig. 6. The
 123 lateral deflection at mid-height was recorded through the use of a linear variable differ-
 124 ential transformer (LVDT). Readings of all the monitored variables and input voltage
 125 were recorded at 1 s intervals using the data acquisition equipment DATASCAN and
 126 logged using the DSLOG software.

127 The global imperfection amplitude, ω_i , consisting of both member bow imperfec-
 128 tion and load eccentricity, can be calculated from the strain gauge readings using Eq.
 129 1 (Gkantou et al. 2016), where ψ is the ratio between the strains on the convex and
 130 concave sides of the cross-section, ω is the 2nd-order eccentricity (i.e. the lateral de-
 131 flection) recorded by the LVDT at mid-height, and I , A and h are the second moment
 132 of area, cross-sectional area and depth of the specimens, respectively. Prior to testing,
 133 the effective imperfection (out-of-straightness plus load eccentricity) in all specimens
 134 was adjusted to achieve as close as possible to $I_{cr}/1000$ by means of two eccentricity
 135 adjusting devices positioned at each end of the members, as shown in Fig. 5. Local
 136 (plate) geometric imperfections were not considered in the current study owing to the
 137 relatively stocky geometries of the cross-sections that are insensitive to local buckling.

$$\omega_i = \frac{2I(1 - \psi)}{Ah(1 + \psi)} - \omega \quad (1)$$

138 All the specimens exhibited a global buckling failure mode as displayed in Fig.
 139 5. The achieved ultimate load, N_u , in each member is reported in Table 3. The load-
 140 deformation relationships obtained from the tests are shown in Figs. 7 - 10, where the
 141 vertical axis is the applied axial load and the horizontal axis is the lateral deflection
 142 recorded by the LVDT at mid-height, ω .

143 **Load-lateral deflection response of HSS columns**

144 The load-lateral deflection curves of the tested specimens can be assessed in rela-
145 tion to the theoretical response as governed by the elastic buckling load N_{cr} , the yield
146 load N_y , the second order elastic behavior and the second order rigid plastic behavior.
147 To illustrate this, examples are given in Figs. 11a and 11b, showing the test results of
148 the C1L1 and C1L6 members, respectively, plotted with the corresponding theoretic-
149 al models, which are described as followed. In relatively slender members (e.g. the
150 C1L6 member shown in Fig. 11b), elastic buckling tends to govern the member fail-
151 ure, whereas the resistance of stockier members (e.g. the C1L3 member in Fig. 11a)
152 is dominated by yielding.

153 The second order elastic and second order rigid plastic models can be used to trace
154 the load-displacement equilibrium path of a compressive member. The second order
155 elastic curve describes the load-lateral deflection response of a compressive member
156 with an assumed initial sinusoidal imperfection, and can be expressed using Eq. 2. In
157 Figs. 11a and 11b, the second order elastic paths were plotted based on the measured
158 imperfections.

$$N = N_{cr} \left(\frac{\omega}{\omega + \omega_i} \right) \quad (2)$$

159 The second order rigid plastic model assumes that a plastic hinge is formed at the
160 mid-height of the member, as shown in Fig. 12. Under such deformation, the hinge will
161 resist both the compressive force N and the second order bending moment $N(\omega + \omega_i)$
162 at mid-height. By assuming a full plastic stress distribution across the cross-section,
163 as illustrated in Fig. 13, where the compression is resisted by the central region (C2)
164 while the moment is taken by the two outer regions (C1 and T3), a relationship between
165 the mid-height deflection and the applied compressive force can be established. As can
166 be seen in Figs. 11a and 11b, the test results are very well captured by the envelope

167 of the two second order boundaries, with the second order elastic model fitting the
 168 initial elastic response of the test specimens and the post-ultimate path merging into
 169 the second order rigid plastic curve. Similar agreement was also achieved for the other
 170 tested specimens. The column test results are employed in the validation of numerical
 171 models in the next section.

172 **NUMERICAL STUDY**

173 **Modeling assumptions**

174 Numerical modeling was carried out firstly to replicate the experimental results,
 175 and subsequently to generate parametric results. The finite element analysis pack-
 176 age ABAQUS (2014) was employed throughout the study. A fine mesh of three-
 177 dimensional four-noded, reduced integration shell element (S4R) was adopted for all
 178 the models. The flat parts of the cross-sections had an element size of the wall thick-
 179 ness t and the corner regions were meshed with a constant number of 5 elements.

180 The measured material and geometric properties were incorporated into the numer-
 181 ical models for the validation study. Given that no significant difference was observed
 182 in the stress-strain responses of the flat and corner material, the average flat coupon test
 183 results, as reported in Table 1, were employed in the FE models. For shell elements,
 184 ABAQUS requires the measured engineering stress-strain curve to be transferred into
 185 true stress-log plastic strain before inputting into the model. The true stress, σ_{true} , and
 186 logarithmic plastic strain, $\varepsilon_{\text{ln}}^{\text{pl}}$, can be obtained using Eqs. 3 and 4, respectively, where
 187 E is the Young's modulus, σ_{nom} is the engineering stress and ε_{nom} is the engineering
 188 strain.

$$\sigma_{\text{true}} = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}) \quad (3)$$

$$\varepsilon_{\text{ln}}^{\text{pl}} = \ln(1 + \varepsilon_{\text{nom}}) - \frac{\sigma_{\text{nom}}}{E} \quad (4)$$

189 The pin-ended boundary conditions employed in the tests were replicated in the
190 numerical models, and each model was assigned an initial geometric imperfection in
191 the form of the first global eigenmode with the amplitude reported in Table 3. The
192 residual stress pattern illustrated in Fig.3, developed based on the measured residual
193 stress amplitudes (Fig. 2), was also applied to the models. Owing to their low mag-
194 nitudes, models without residual stresses were also simulated to assess their influence.
195 Considering both geometrical and material nonlinearities, the FE models were solved
196 by means of the modified Riks method (ABAQUS 2014), allowing the pre- and post-
197 ultimate behavior of the columns to be traced.

198 **Validation of FE models**

199 The ability of the numerical models to capture accurately the behavior observed
200 in the experiments was assessed throughout a series of comparisons between the test
201 and FE results. First, the experimental ultimate loads were compared to those obtained
202 from the numerical models with and without residual stresses, as shown in Table 4.
203 Typical failure modes of the test specimens and FE models are given in Fig. 14, while
204 typical load-lateral deflection curves derived from the FE models (for members C1L5
205 and C5L3) are plotted against the test results in Figs. 15a and 15b, respectively. As
206 can be seen in Table 4 and from Figs. 15a and 15b, the FE models with and without
207 residual stresses were both able to capture accurately the flexural buckling response of
208 the column specimens, with an average error of 1.5% and 1.7%, respectively, in the
209 ultimate load predictions. Given the very low magnitude of the residual stresses and
210 their minimal influence on the member strengths, FE models without residual stresses
211 were used in the parametric investigation.

212 **FE parametric studies**

213 Following successful validation of the FE models, a series of parametric studies
214 were carried out. A total of 144 models were generated, covering two steel grades

215 (S460 and S690), two cross-section aspect ratios (150×150 and 250×150), three cross-
216 section slendernesses (Class 1, 2 and 3 according to the cross-section classification
217 limits given in Eurocode 3 (CEN 2005)), eight column slendernesses (0.45 - 2.5), and
218 two buckling axes (minor and major) for the 250×150 section sizes. The input ma-
219 terial parameters in the S460 and S690 FE models were defined based on the coupon
220 test results of S460 SHS $50 \times 50 \times 5$ and S690 SHS $50 \times 50 \times 5$, respectively. In the non-
221 linear analyses, geometric imperfections were adopted in the form of the first global
222 eigenmode (approximately a half-sine wave), with an amplitude of $L_{cr}/1000$. This im-
223 perfection amplitude was employed in the formulation of the European, Chinese and
224 Australian column buckling curves (CEN 2005; MOHURD 2003; Standards Australia
225 1998), while a smaller value of $L_{cr}/1500$ was adopted in the development of the AISC
226 buckling curves (AISC 2010). The generated parametric results are used, in conjunc-
227 tion with the test results, to assess the applicability of different codified buckling curves
228 to the design of hot-finished HSS SHS and RHS columns.

229 **DESIGN RECOMMENDATIONS**

230 The current column design rules in the European, North American, Chinese and
231 Australian structural steel design codes (CEN 2005; AISC 2010; MOHURD 2003;
232 Standards Australia 1998) are introduced in this section. Based on the obtained test and
233 numerical results, reliability analyses are carried out to examine the suitability of the
234 presented design provisions for hot-finished S460 and S690 SHS and RHS columns.

235 **Current design provisions**

236 The compressive strength (flexural buckling resistance) of a column, N_b , is typi-
237 cally calculated by multiplying the plastic resistance (yield load) of the cross-section,
238 Af_y , with a buckling reduction factor, χ , to account for member instability, as shown
239 in Eq. 5. The relationships between this reduction factor and the member slenderness,
240 generally referred to as buckling curves, vary between standards, as described in this

241 section. It should be noted that all the cross-sections in this study are classified as
 242 “fully effective” in the context of the design codes considered, and the case of slender
 243 (Class 4) cross-sections, in which local buckling prevents the attainment of the yield
 244 load, is out of the scope of this paper.

$$N_b = \chi A f_y \quad \text{for fully effective cross-sections} \quad (5)$$

245 *European Standard (EC 3)*

246 In Eurocode 3 (CEN 2005), the relative member slenderness, denoted $\bar{\lambda}_{EC}$ herein,
 247 is defined by Eq. 6, where i is the radius of gyration about the relevant axis, and other
 248 symbols are as previously defined.

$$\bar{\lambda}_{EC} = \sqrt{\frac{A f_y}{N_{cr}}} = \frac{L_{cr}}{i \pi} \sqrt{\frac{f_y}{E}} \quad (6)$$

249 Eurocode 3 (CEN 2005) employs the multiple column curve concept (Sfintesco
 250 1970; Jacquet 1970; Beer and Schulz 1970) and, using the Ayrton-Perry (Ayrton and
 251 Perry 1886) formula, defines a set of five buckling curves through five discrete values
 252 of the imperfection factor α , as given in Eqs. 7 and 8. It should be noted that in Eq.8,
 253 the term $\alpha(\bar{\lambda}_{EC} - 0.2)$ is the imperfection term η , which was taken on the basis of first
 254 yield as $\eta = h \Lambda \omega_i / 2I$ in the original Ayrton-Perry (Ayrton and Perry 1886) expression,
 255 with h , A , ω_i and I being as previously defined. The buckling curve selection depends
 256 on the cross-section shape, buckling axis, steel grade and manufacturing route of the
 257 member. For hot-finished S460 SHS and RHS, buckling curve a_0 with an imperfection
 258 factor of 0.13 is specified in EN 1993-1-1 (CEN 2005). For higher steel grades, up
 259 to S700, EN 1993-1-12 (CEN 2007) states that the buckling curves for S460 material
 260 should be adopted. Comparison between the Eurocode 3 buckling curve a_0 and the
 261 test/FE data of both steel grades is shown in Fig. 16, where the vertical axis for the
 262 test/FE data is the ultimate compressive resistance normalized by the cross-sectional

263 plastic resistance Af_y .

$$\chi_{EC} = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}_{EC}^2}} \quad (7)$$

$$\Phi = 0.5[1 + \alpha(\bar{\lambda}_{EC} - 0.2) + \bar{\lambda}_{EC}^2] \quad (8)$$

264 *North American Standard (AISC 360)*

265 In AISC 360 (AISC 2010), the relative member slenderness $\bar{\lambda}_{AISC}$ is defined as in
266 Eurocode 3, though with slightly different notation, as given by Eq. 9. In AISC 360,
267 K is the effective length factor, with KL equivalent to the member buckling length L_{cr}
268 in Eurocode 3, while r is symbol adopted for radius of gyration and F_y is the material
269 yield strength.

$$\bar{\lambda}_{AISC} = \frac{KL}{\pi r} \sqrt{\frac{F_y}{E}} = \frac{L_{cr}}{i\pi} \sqrt{\frac{f_y}{E}} \quad (9)$$

270 AISC 360 (AISC 2010) adopts a single column curve which was developed from
271 the SSRC (Structural Stability Research Council) column curves (Bjorhovde and Tall
272 1971; Bjorhovde 1972, 1978; Ziemian 2010). The single column curve consists of two
273 basic expressions for the buckling reduction factor χ_{AISC} - an exponential expression
274 (Eq. 10) in the inelastic range accounting for the effect of residual stresses, and a
275 reduced Euler expression (Eq. 11) in the elastic range where the residual stresses
276 are believed to have minimal influence (Tide 1985). The constant terms in the two
277 equations were determined based on the test data and reliability criteria at the time
278 (Tide 1985, 2001; Beedle 1991). The AISC column curve is also plotted in Fig. 16.

$$\chi_{AISC} = 0.658^{\bar{\lambda}_{AISC}^2} \quad \text{for } \bar{\lambda}_{AISC} \leq 1.5 \quad (10)$$

$$\chi_{\text{AISC}} = \frac{0.877}{\bar{\lambda}_{\text{AISC}}^2} \quad \text{for } \bar{\lambda}_{\text{AISC}} > 1.5 \quad (11)$$

279 *Chinese Standard (GB 50017)*

280 The relative member slenderness in the Chinese code GB 50017 (MOHURD 2003)
 281 $\bar{\lambda}_{\text{GB}}$ is defined in the same manner as in EC 3 (CEN 2005) and AISC (AISC 2010), as
 282 given by Eq. 12.

$$\bar{\lambda}_{\text{GB}} = \frac{L_{\text{cr}}}{i\pi} \sqrt{\frac{f_y}{E}} \quad (12)$$

283 In GB 50017 (MOHURD 2003), a set of four buckling curves are employed. These
 284 curves were derived based on computational modeling of columns with various cross-
 285 section types (Li and Xiao 1982; Li et al. 1985), and are described by an Ayrton-Perry
 286 formula (Ayrton and Perry 1886; Luo 1989), as given in Eqs. 13 and 14, where the
 287 Ayrton-Perry imperfection term $\eta = \alpha_3 \bar{\lambda}_{\text{GB}} + \alpha_2 - 1$, and α_1 , α_2 and α_3 are factors
 288 that depend on the selected buckling curve. For steel strengths up to 420 N/mm²,
 289 which is the maximum strength covers by GB 50017, buckling curve b is specified for
 290 hot-finished SHS and RHS with $\alpha_1 = 0.65$, $\alpha_2 = 0.965$ and $\alpha_3 = 0.300$. The results
 291 of the reliability analysis presented in the next section shows that, as in the European
 292 Standard, a higher buckling curve can be used for high strength material, and curve a
 293 with $\alpha_1 = 0.41$, $\alpha_2 = 0.986$ and $\alpha_3 = 0.152$ is recommended herein for hot-finished
 294 HSS SHS and RHS of grade S460 and above.

$$\chi_{\text{GB}} = 1 - \alpha_1 \bar{\lambda}_{\text{GB}}^2 \quad \text{for } \bar{\lambda}_{\text{GB}} \leq 0.215 \quad (13)$$

$$\chi_{GB} = \frac{1}{2\bar{\lambda}_{GB}^2} \left[(\alpha_2 + \alpha_3\bar{\lambda}_{GB} + \bar{\lambda}_{GB}^2) - \sqrt{(\alpha_2 + \alpha_3\bar{\lambda}_{GB} + \bar{\lambda}_{GB}^2)^2 - 4\bar{\lambda}_{GB}^2} \right] \text{ for } \bar{\lambda}_{GB} > 0.215 \quad (14)$$

295 *Australian Standard (AS 4100)*

296 The AS 4100 relative member slenderness $\bar{\lambda}_{AS}$ is equivalent to $\bar{\lambda}_{EC}$ multiplied by
 297 a factor $\pi\sqrt{E/250}$, as defined in Eq. 15 (Standards Australia 1998).

$$\bar{\lambda}_{AS} = \frac{L_{cr}}{i} \sqrt{\frac{f_y}{250}} = \frac{L_{cr}}{i\pi} \sqrt{\frac{f_y}{E}} \left(\pi\sqrt{\frac{E}{350}} \right) \quad (15)$$

298 The AS 4100 (Standards Australia 1998) also adopts the multiple column curve
 299 concept. A set of five buckling curves developed from the SSRC column curves
 300 (Bjorhovde and Tall 1971; Bjorhovde 1972, 1978; Ziemian 2010) are defined in an
 301 Ayrton-Perry (Ayrton and Perry 1886) format. The AS 4100 buckling curves are ex-
 302 pressed through Eqs. 16-18, where α_a is a slenderness modifier obtained from regres-
 303 sion analysis (Rotter 1982), and α_b is an imperfection factor related to the choice of
 304 the buckling curve (Trahair and Bradford 1998; Rotter 1982). The imperfection term η
 305 in the original Ayrton-Perry formula is replaced by $\eta = 0.00326(\bar{\lambda}_{AS} + \alpha_a\alpha_b - 13.5)$
 306 in the AS 4100 expression. The buckling curve with $\alpha_b = -1.0$, which is currently
 307 specified for hot-rolled and cold-formed normal strength steel SHS, RHS and CHS,
 308 is adopted for comparison with the HSS SHS and RHS column data generated here-
 309 in. This curve has been plotted in Fig. 16, with the slenderness $\bar{\lambda}_{AS}$ being divided by
 310 $\pi\sqrt{E/250}$ to maintain consistency with the other codes.

$$\chi_{AS} = \xi \left\{ 1 - \sqrt{\left[1 - \left(\frac{90}{\xi(\bar{\lambda}_{AS} + \alpha_a\alpha_b)} \right)^2 \right]} \right\} \quad (16)$$

311 where

$$\alpha_a = \frac{2100(\bar{\lambda}_{AS} - 13.5)}{\bar{\lambda}_{AS}^2 - 15.3\bar{\lambda}_{AS} + 2050} \quad (17)$$

312 and

$$\xi = \frac{[(\bar{\lambda}_{AS} + \alpha_a \alpha_b)/90]^2 + 1 + 0.00326(\bar{\lambda}_{AS} + \alpha_a \alpha_b - 13.5)}{2 [(\bar{\lambda}_{AS} + \alpha_a \alpha_b)/90]^2} \quad (18)$$

313 **Reliability analyses and discussion**

314 From Fig. 16, it may be observed that the EC 3 and AS 4100 buckling curves
315 are nearly identical, while the GB 50017 curve gives slightly lower predictions in the
316 intermediate slenderness range, and the AISC 360 one is the lowest of the four. All
317 four curves converge towards the Euler elastic buckling curve in the slender range.

318 In each code, partial factors are applied to the nominal column resistance in order
319 to achieve a specified target reliability. This partial factor is denoted as γ_{M1} in EC
320 3 with the value set to unity. In AISC 360 and AS 4100, the partial safety factor is
321 represented by ϕ_c and ϕ , respectively, with a value of 0.9. Note that γ_{M1} applies to
322 the dominator whereas ϕ_c and ϕ appear in the numerator. The Chinese standard GB
323 50017 implicitly incorporates this factor in the definition of the design yield strength
324 $f_d = f_{nom}/\gamma_M$, where γ_M is dependent on the steel grade and thickness of the material.
325 Since a γ_M value for high strength steel grades is not specified in GB 50017, a value of
326 $\gamma_M = 1.1$, based on the results obtained for S460 and S690 steels from a recent study
327 Shi et al. (2016), is adopted herein for both steel grades (S460 and S690); this value is
328 also close to those specified in GB 50017 for S235-S420 steels. The factored column
329 design curves are compared with the test and FE data in Fig. 17, where the four design
330 curves tend to provide safe-side predictions for the S690 members, while some of the
331 S460 test data fall below the EC 3 curve. On average, the normalized S690 column test
332 data are about 5% higher than the S460 data, but with higher scatter. The normalized

333 S690 FE data lie about 2% above the S460 data.

334 To examine the suitability of the factored design curves, reliability analyses in ac-
335 cordance with EN 1990 (CEN 2002) and AISC 360 (AISC 2010) were carried out. For
336 the Eurocode analysis, the mean to nominal yield strength ratio $f_{y,mean}/f_{y,nom} = 1.135$
337 (i.e. the material over-strength) and coefficient of variation of the yield strength $V_{f_y} =$
338 0.055 were obtained from a series of coupon tests results collected from steel produc-
339 ers (Wang et al. 2016). The coefficient of variation of the geometric properties V_g was
340 taken as 0.02 (Byfield and Nethercot 1997). In the AISC analysis, the $f_{y,mean}/f_{y,nom}$, V_{f_y}
341 and V_g values derived from lower grade carbon steel test data were used (Bartlette et al.
342 2003), with values of 1.028, 0.058 and 0.05, respectively. To add artificial variability
343 to the numerical results, a variability term $V_{FE} = 0.44$, determined by considering
344 the deviation of numerical to experimental results, was incorporated in both analyses,
345 following a similar approach to Davaine (2005) and Bock et al. (2015).

346 The key parameters and results of the Eurocode and AISC reliability analyses are
347 summarized in Tables 5 and 6, respectively. In the Eurocode analysis (Table 5), $k_{d,n}$
348 is the design fractile factor for n data points of the dataset under consideration (CEN
349 2002; Afshan et al. 2015); b is the average ratio of experimental to model resistance
350 based on a least squares fit to the test data; V_δ is the coefficient of variation of the test
351 and FE results relative to the resistance model; V_r is the combined coefficient of varia-
352 tion incorporating both model and basic variable (material and geometry) uncertainties,
353 where the dependence of the basic variables was separated (Afshan et al. 2015), and
354 hence varies between data points, but the average value for each dataset is reported in
355 Tables 5 and 6; γ_{M1} is the required partial factor, which can be assessed against the
356 EC 3 specified value of 1.0. In the AISC analysis (Table 6), V_Q is the coefficient of
357 variation of the load effects, determined based on an assumed live-to-dead load ratio
358 of 3:1 for hot-rolled sections (AISC 2010); V_R is equivalent to and calculated in the
359 same way as the V_r parameter in the Eurocode analysis; β is the reliability index with

360 a target value greater than 2.6 required by AISC 360 for member design.

361 From Table 5, it may be seen that, using curve a_0 , the required values of γ_{M1}
362 indicated by the statistical analyses are 1.12 and 1.11 for the S460 and S690 material,
363 respectively considering both the test and FE results. Based on the tests only, slightly
364 higher values are obtained. From these results, which accord with those of Charbrolin
365 (2002) determined during the development of EC 3, it may be concluded that a higher
366 value of γ_{M1} is required in order to meet the Eurocode reliability requirements. A value
367 of $\gamma_{M1} = 1.1$ (for both normal strength and high strength steels) is recommended
368 herein, and is consistent with the partial factors employed in the North American,
369 Chinese, and Australian codes. The AISC curve is also found to be satisfactory for
370 both steel grades, as shown in Table 6, where β is equal or greater than 2.6 in all cases.
371 The AS 4100 and GB 50017 design curves gave more conservative predictions than
372 Eurocode 3, and hence may be considered to yield acceptable reliability. Overall, it is
373 concluded that the four selected buckling curves from the European, North American,
374 Chinese and Australian codes can be safely applied to the design of hot-finished HSS
375 SHS and RHS columns, and while the S690 columns perform slightly better than the
376 S460 columns, a separate (higher) buckling is not considered to be warranted at this
377 stage.

378 CONCLUSIONS

379 A comprehensive experimental and numerical study into the flexural buckling be-
380 havior of hot-finished high strength steel (HSS) SHS and RHS columns has been car-
381 ried out in this paper. A total of 30 pin-ended columns have been tested, covering both
382 S460 and S690 steels, five SHS cross-section sizes, and eight member slendernesses
383 (0.45-2.25). The effective global imperfection was adjusted to achieve a value of ap-
384 proximately $L_{cr}/1000$ in all test specimens. Incorporating the measured material and
385 geometric properties, FE models have been developed, and shown to be able to accu-

386 rately replicate the experimental results. Parametric studies considering both SHS and
387 RHS with various member geometries followed, leading to the generation of a total
388 of 144 FE results. The S690 columns showed improved normalized buckling perfor-
389 mance over the S460 columns by about 5% in the tests and 2% in the models. Based
390 on the test and numerical results, reliability analyses in accordance with EN 1990 and
391 AISC 360 were carried out, showing that the current HSS column design curves in the
392 European standard (with $\gamma_{M1} = 1.1$), and those selected from the North American,
393 Chinese and Australian standards, are applicable to hot-finished HSS SHS and RHS
394 columns.

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398 S.

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TABLE 1: Average measured flat coupon results for each cross-section size

Cross-section	E (N/mm ²)	f_y (N/mm ²)	f_u (N/mm ²)	ε_f (%)	f_u/f_y
S460 SHS 50×50×5	211000	505	620	31.0	1.23
S460 SHS 70×70×6.3	212000	531	752	26.3	1.41
S460 SHS 100×100×5	211000	511	616	29.2	1.21
S690 SHS 50×50×5	206000	759	790	21.7	1.04
S690 SHS 100×100×5.6	210000	782	798	19.2	1.02

TABLE 2: Measured tensile coupon test results

Cross-section	Label	E (N/mm ²)	f_y (N/mm ²)	f_u (N/mm ²)	ε_f (%)	f_u/f_y
S460 SHS 50×50×5	F1	211000	494	618	31.5	1.25
	F2	211000	506	620	30.0	1.23
	F3	211000	515	623	31.4	1.21
	C	208000	481	631	26.2	1.31
S460 SHS 70×70×6.3	F1	212000	529	757	26.6	1.43
	F2	212000	534	744	27.2	1.39
	F3	215000	542	769	26.2	1.42
	F4	209000	520	736	25.0	1.42
S460 SHS 100×100×5	F1	211000	515	618	30.5	1.20
	F2	212000	507	615	27.8	1.21
	C	208000	528	636	23.3	1.20
S690 SHS 50×50×5	F1	200000	749	783	20.3	1.05
	F2	210000	776	800	19.1	1.03
	F3	205000	761	795	19.8	1.04
	F4	202000	750	781	20.2	1.04
	C	210000	782	813	19.2	1.03
S690 SHS 100×100×5.6	F1	212000	794	803	20.0	1.01
	F2	209000	770	793	18.4	1.03
	C	209000	774	792	20.2	1.02

TABLE 3: Measured geometric dimensions and key test results of column specimens

Cross-section	Label	L_{cr} (mm)	h (mm)	b (mm)	t (mm)	r_i (mm)	ω_i (mm)	$\bar{\lambda}$	N_u (kN)
S460 SHS 50×50×5	C1L1	427.0	50.33	50.32	4.98	2.02	0.42	0.36	427
	C1L2	668.5	50.23	50.36	4.69	2.31	0.70	0.57	396
	C1L3	907.0	50.48	50.44	4.95	2.05	0.93	0.77	384
	C1L4	1220.0	50.26	50.36	4.63	2.37	1.16	1.03	282
	C1L5	1529.0	50.43	50.43	4.89	2.11	1.45	1.30	217
	C1L6	1700.0	50.37	50.52	5.01	2.00	1.75	1.44	182
	C1L7	1859.0	50.32	50.32	5.05	1.95	1.86	1.58	151
	C1L8	2150.0	50.37	50.39	4.92	2.08	2.21	1.83	126
S460 SHS 70×70×6.3	C2L1	649.5	70.00	69.96	6.22	3.78	0.64	0.40	792
	C2L2	939.0	69.90	69.95	6.29	3.72	0.94	0.59	762
	C2L3	1280.0	69.99	69.97	6.37	3.63	1.17	0.80	651
	C2L4	1710.0	69.83	69.91	6.32	3.68	1.80	1.07	531
	C2L5	2150.0	69.96	70.06	6.32	3.69	2.34	1.34	367
	C2L6	2400.0	69.95	70.02	6.21	3.79	2.53	1.49	309
	C2L7	2600.0	69.95	70.07	6.17	4.34	2.67	1.62	264
	C2L8	3020.0	70.00	70.02	6.37	3.63	3.08	1.88	208
S460 SHS 100×100×5	C3L1	858.3	99.69	99.28	5.19	5.81	0.91	0.35	878
	C3L2	1759.0	99.82	99.28	5.31	5.69	1.73	0.72	798
	C3L3	2949.0	99.37	99.82	5.23	5.00	2.24	1.22	557
S690 SHS 50×50×5	C4L1	426.0	50.47	50.44	4.99	2.02	0.48	0.44	690
	C4L2	668.5	50.47	50.47	4.76	2.24	0.71	0.69	637
	C4L3	905.5	50.45	50.43	4.82	2.18	0.93	0.94	562
	C4L4	1220.0	50.67	50.51	4.79	2.21	1.18	1.26	391
	C4L5	1529.0	50.40	50.40	4.79	2.21	1.60	1.59	248
	C4L6	1700.0	50.60	50.40	4.95	2.05	1.72	1.76	201
	C4L7	1860.0	50.53	50.48	4.93	2.07	1.77	1.93	166
	C4L8	2150.0	50.60	50.52	4.84	2.16	2.04	2.22	119
S690 SHS 100×100×5.6	C5L1	858.0	100.43	100.53	5.67	5.33	1.03	0.44	1571
	C5L2	1760.0	100.50	100.52	5.72	4.78	1.66	0.89	1420
	C5L3	2950.0	100.70	100.59	5.78	6.22	3.00	1.50	680

TABLE 4: Comparison of column test results with FE results

Cross-section	Label	$N_{u,\text{test}}$ (kN)	With residual stresses		Without residual stresses	
			$N_{u,\text{FE}}$ (kN)	$N_{u,\text{FE}}/N_{u,\text{test}}$	$N_{u,\text{FE}}$ (kN)	$N_{u,\text{FE}}/N_{u,\text{test}}$
S460 SHS 50×50×5	C1L1	427	427	1.00	427	1.00
	C1L2	396	391	0.99	391	0.99
	C1L3	384	388	1.01	388	1.01
	C1L4	282	298	1.06	300	1.07
	C1L5	217	226	1.04	226	1.04
	C1L6	182	189	1.04	190	1.04
	C1L7	151	161	1.06	161	1.06
	C1L8	126	121	0.96	121	0.96
S460 SHS 70×70×6.3	C2L1	792	781	0.99	782	0.99
	C2L2	762	767	1.01	767	1.01
	C2L3	651	739	1.14	741	1.14
	C2L4	531	564	1.06	567	1.07
	C2L5	367	396	1.08	401	1.09
	C2L6	309	323	1.05	323	1.05
	C2L7	264	278	1.06	279	1.06
	C2L8	208	217	1.04	217	1.05
S460 SHS 100×100×5	C3L1	878	939	1.07	939	1.07
	C3L2	798	896	1.12	893	1.12
	C3L3	557	581	1.04	584	1.05
S690 SHS 50×50×5	C4L1	690	642	0.93	643	0.93
	C4L2	637	584	0.92	589	0.92
	C4L3	562	523	0.93	523	0.93
	C4L4	391	357	0.91	357	0.91
	C4L5	248	234	0.94	234	0.95
	C4L6	201	198	0.99	198	0.99
	C4L7	166	166	1.00	166	1.00
	C4L8	119	125	1.05	125	1.05
S690 SHS 100×100×5.6	C5L1	1571	1576	1.00	1571	1.00
	C5L2	1420	1402	0.99	1407	0.99
	C5L3	680	676	0.99	676	0.99
Mean				1.015		1.017
COV				0.055		0.055

TABLE 5: Summary of statistical parameters for the Eurocode reliability analysis

Dataset	n	$k_{d,n}$	b	V_δ	V_r	γ_{M1}
S460 test	19	3.70	0.967	0.038	0.060	1.16
S460 test+FE	91	3.20	1.017	0.025	0.063	1.12
S690 test	11	4.33	1.053	0.054	0.070	1.14
S690 test+FE	83	3.21	1.032	0.027	0.065	1.11

TABLE 6: Summary of statistical parameters for the AISC reliability analysis

Dataset	n	V_Q	V_R	ϕ_c	β
S460 test	19	0.19	0.060	0.9	2.58
S460 test+FE	91	0.19	0.063	0.9	2.76
S690 test	11	0.19	0.070	0.9	3.09
S690 test+FE	83	0.19	0.065	0.9	3.02

511 FIGURES

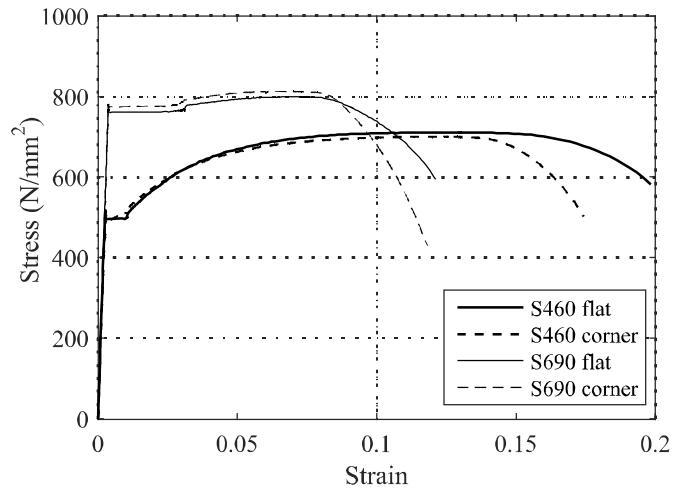


FIG. 1: Typical measured stress-strain curves of S460 and S690 flat and corner coupons

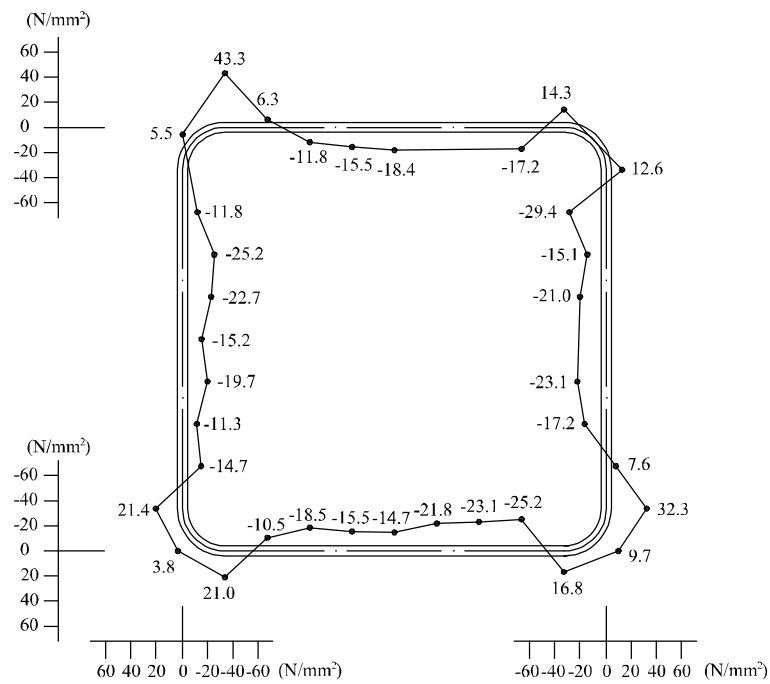


FIG. 2: Measured residual stress distribution in S690 SHS 90 × 90 × 5.6. Residual stresses are shown in N/mm², with positive values being tensile and negative values compressive

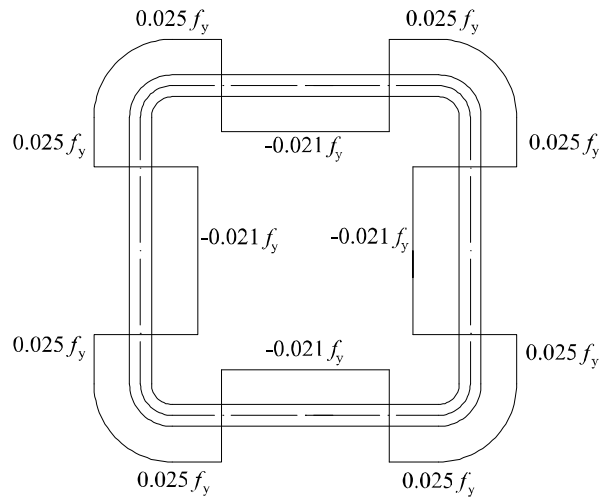


FIG. 3: Residual stress distribution incorporated into FE models.

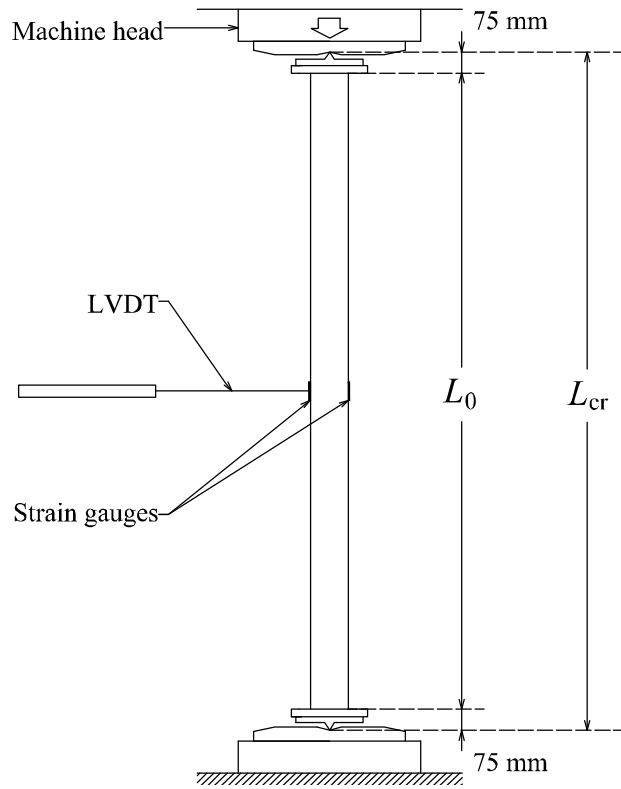


FIG. 4: Column test setup.

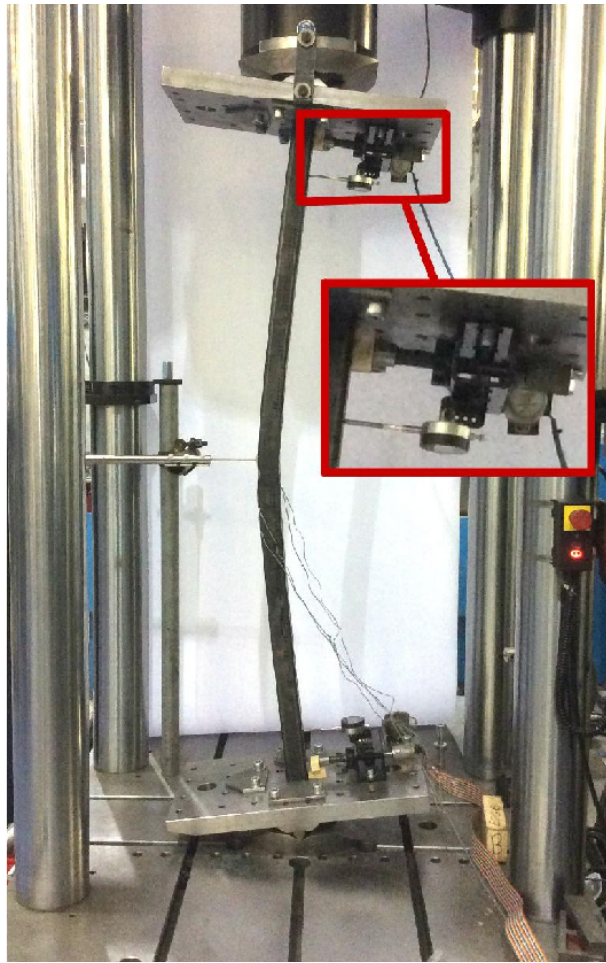


FIG. 5: Photo of column test setup and typical failure mode of specimen

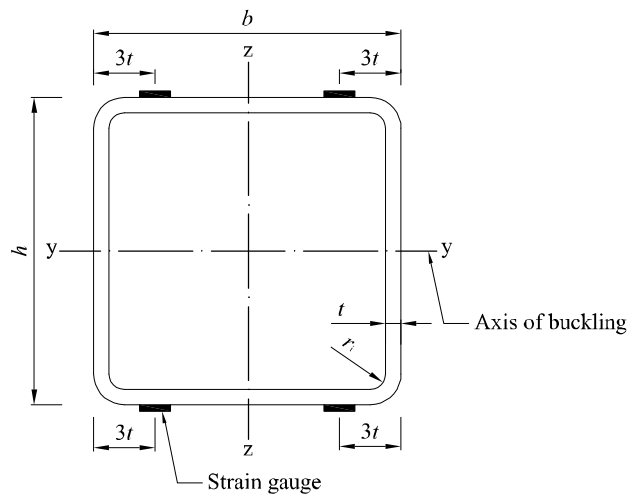


FIG. 6: Geometry of tested cross-sections and locations of strain gauges

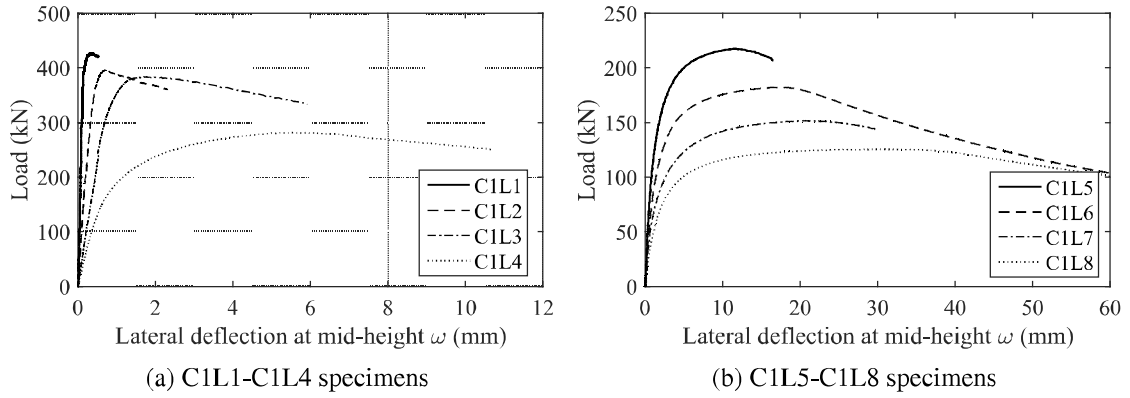


FIG. 7: Load-lateral displacement curves of C1 (S460 SHS 50x50x5) specimens

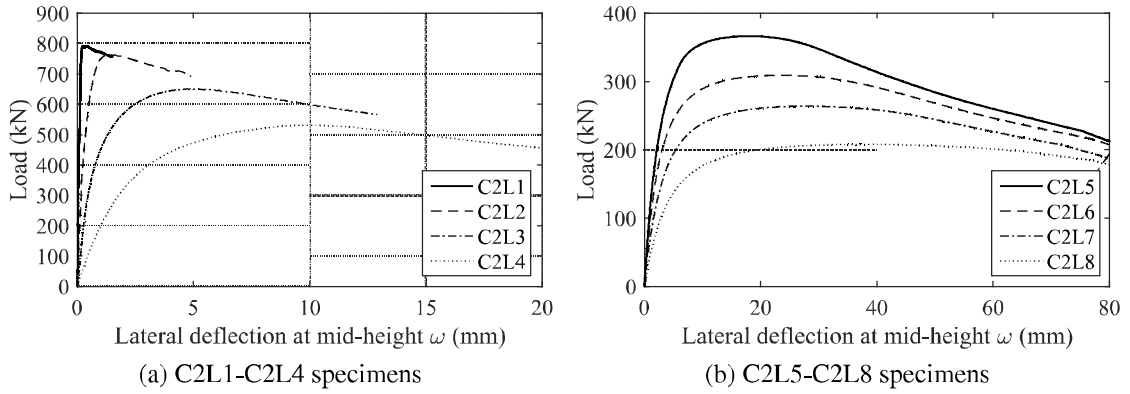


FIG. 8: Load-lateral displacement curves of C2 (S460 SHS 70x70x6.3) specimens

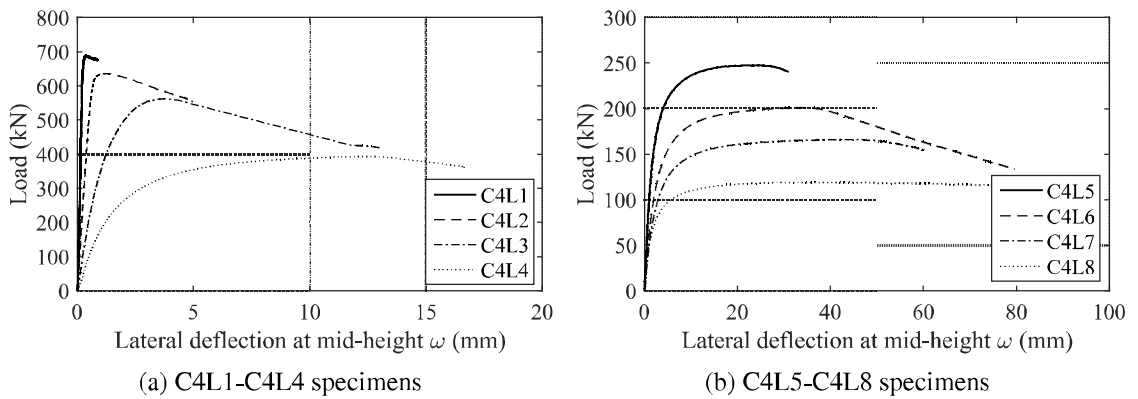


FIG. 9: Load-lateral displacement curves of C4 (S690 SHS 50x50x5) specimens

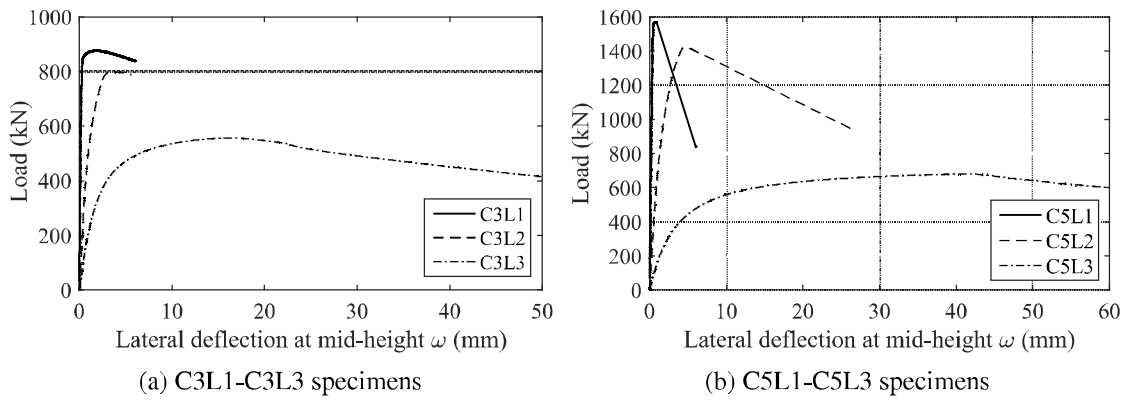


FIG. 10: Load-lateral displacement curves of a) C3 (S460 SHS 100×100×5) and b) C5 (S690 SHS 100×100×5.6) specimens

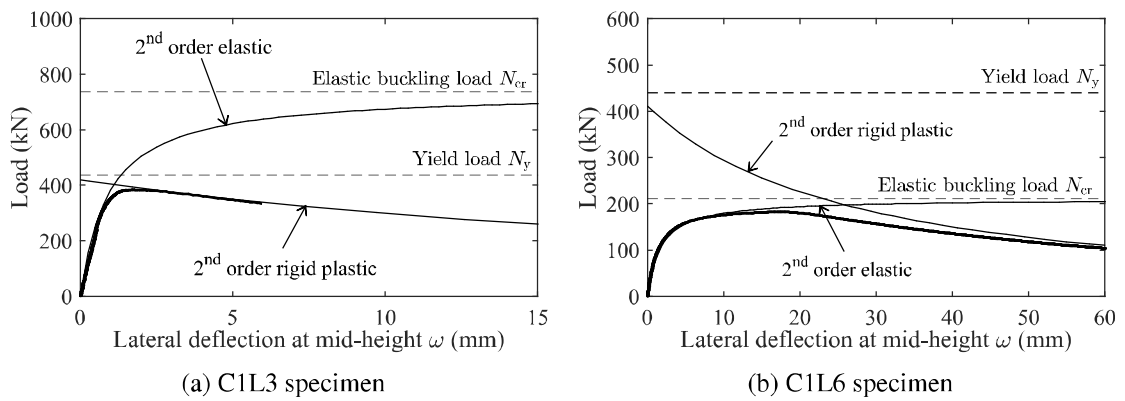


FIG. 11: Load-lateral displacement curves of a) C1L3 and b) C1L6 specimens and comparison with theoretical predictions

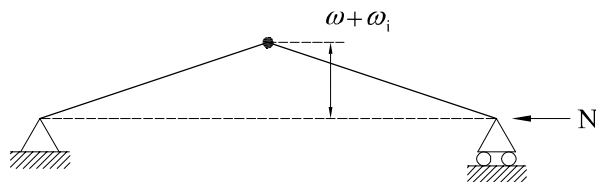


FIG. 12: Second order rigid plastic model

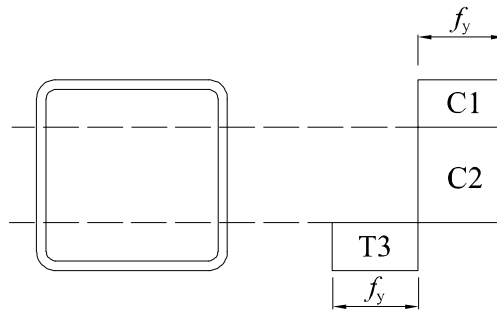


FIG. 13: Assumed plastic stress distribution



(a) Experiment



(b) FE model

FIG. 14: Comparison of the failure modes in tested specimen and numerical model (C1L5 member)

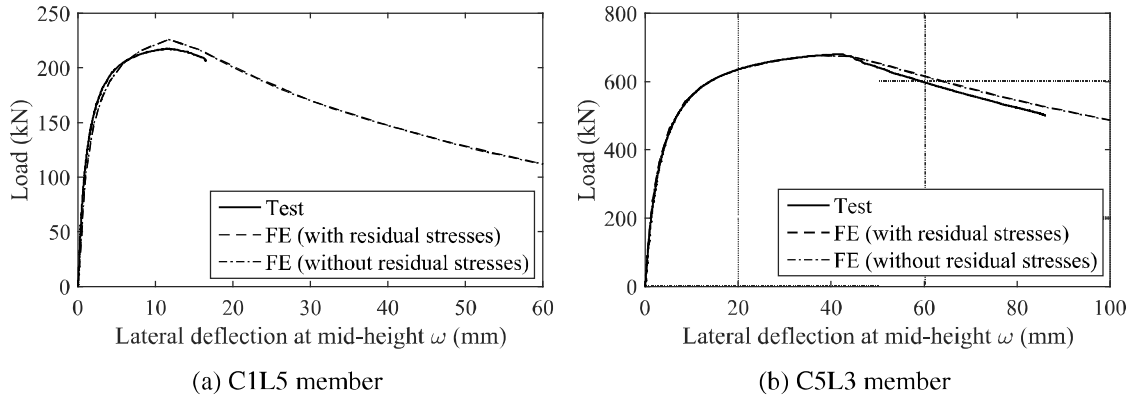


FIG. 15: Comparison between the load-lateral displacement curves obtained from the test and FE models of a) C1L5 and b) C5L3 member

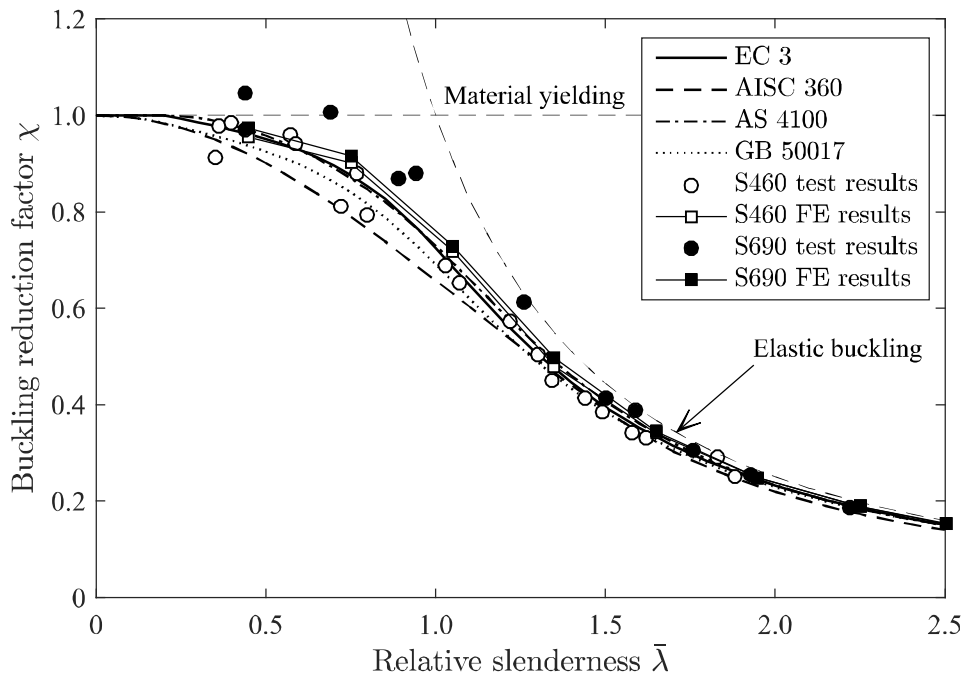


FIG. 16: Normalized test and FE results with nominal column buckling curves

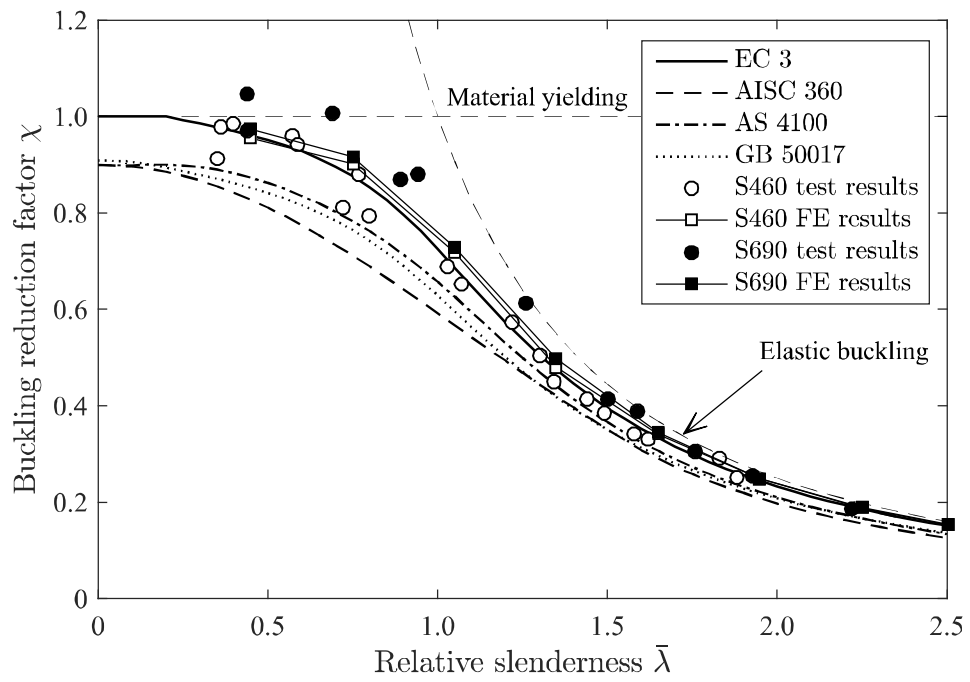


FIG. 17: Normalized test and FE results with design column buckling curves