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1 **Local buckling resistances of cold-formed high-strength steel SHS and** 2 **RHS with varying corner radius**

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9 **Abstract:**

10 This paper reports an experimental and numerical study of the cross-sectional behaviour of
11 cold-formed high-strength steel (HSS) square and rectangular hollow section (SHS and RHS)
12 members. Six stub column tests, six 3-point and three 4-point bending tests have been carried
13 out, followed by the development of accurate finite element (FE) models and a parametric
14 study. Based on the test and FE results, the design rules for slender cross-sections in European,
15 American, Australian and Chinese standards and the GSRM method have been assessed via
16 reliability analyses. A new equation for deriving the cross-sectional slenderness of SHS and
17 RHS considering the influence of corner radius and new area reduction factors for designing
18 slender cold-formed S900 (and above) SHS and RHS have also been proposed.

19 **Keywords:** Cold-formed steel structures, Effect of corner radius, High strength steels, Local
20 buckling, Square and rectangular hollow sections (SHS and RHS), Stub column tests, 3-point
21 bending tests, 4-point bending tests.

22 **1. Introduction**

23 High-strength steels (HSS) are defined as steels with nominal yielding strengths greater than

24 460 MPa (and up to 1100 MPa) and are currently widely available in the market. Their superior
25 strength to weight ratio has motivated their wide application, as can be evidenced by the
26 increasing number of landmark constructions worldwide [1][2]. Meanwhile, the design rules
27 for structural HSS are being developed, which mainly involve the assessments of the
28 applicability of current design rules that were originally designed for ordinary strength steels
29 (e.g. EN 1993-1-1 [3], AISC 360-16 [4], AS 4100 [5] and GB 50017 [6]) on high strength steel
30 members. The European standard, EN 1993-1-12 [7], serves as one of the only few official
31 design provisions that cover the design of HSS up to S700, and the design against stability is
32 largely a simple extension of the design rules in EN 1993-1-1 [3] set out for conventional steel
33 structures.

34 Focusing on the design of cold-formed HSS square and rectangular hollow sections (SHS and
35 RHS), a review of relevant literatures is given herein. Ma et. al. has carried out a series of
36 experimental and numerical studies on S700 and S900 SHS and RHS stub columns [8][9] and
37 beams under 4-point bending [10][11]. Compared to the 4-point bending test and FE data [11],
38 the plastic and yield slenderness limits set out in EN 1993-1-1 [3], AISC 360-16 [4] and AS
39 4100 [5] were shown to be conservative, and new slenderness limits were proposed [11]. For
40 the design of slender (Class 4) cross-sections, the Effective Width Method (EWM) [12] and the
41 Direct Strength Method (DSM) [13] were shown to overestimate slightly the axial resistances
42 of S700, S900 and S1100 SHS and RHS [9] while underestimate significantly their moment
43 capacities [11]. To this end, new safety factors were proposed for the design of slender cold-
44 formed HSS SHS and RHS stub columns [9] and a less conservative DSM equation adopting
45 modified parameters was proposed for designing the beams [11]. Feldmann et. al. [14] have

46 also carried out a series of stub column tests on cold-formed S500, S700 and S960 SHS and
47 found that the EWM [12] was slightly unsafe compared to the test data, which was in an
48 agreement with the observations made by Ma et. al. [8][9]. They have also pointed out that how
49 the original plate width is taken in SHS and RHS has a great impact on the result, and hence
50 suggested to use the whole cross-section slenderness, as can be obtained by finite strip method
51 [13][15], for the assessment of design methods for slender SHS and RHS.

52 The material properties and residual stresses of cold-formed S700 and S900 SHS and RHS
53 members were studied in Ma et. al. [16], where the bending residual stresses were measured as
54 40% - 60% of the 0.2% proof strength on average. These were significantly larger than the
55 measured membrane residual stresses which reported a maximum value of only 20% of the 0.2%
56 proof strength. The magnitudes of residual stresses were also shown to be higher in the
57 transverse direction than in the longitudinal direction [16]. Somodi & Kovesdi [17] also carried
58 out residual stress measurements on cold-formed SHS and RHS across a larger range of steel
59 grades (S400, S420, S500, S700 and S960), and drew similar conclusions as in Ma et. al. [16]
60 that the longitudinal membrane residual stresses in cold-formed HSS SHS and RHS are
61 negligible. For this reason, only characterisation models for the bending residual stresses were
62 proposed [17]. In the subsequent research on the overall structural performance of these cold-
63 formed sections, as in Ma et al. [9][11] and Somodi & Kovesdi [18], the longitudinal membrane
64 residual stresses were ignored in the numerical models developed.

65 In summary, the research works reviewed above have investigated the local buckling behaviour
66 of cold-formed HSS SHS and RHS and proposed residual stress models and cross-sectional

67 design rules. However, in these studies, the influence of corner geometry on the resistance of
68 cold-formed HSS tubular sections has not received enough attention. It was found that as the
69 steel grade increases, the inner corner radius required for the cold-forming process also
70 increases, from as low as $0.5t$ in S500 steels to as high as $3.5t$ in S900 and S1100 steels [19].
71 This will render the current assumption for SHS and RHS design in different standards
72 inaccurate: simply taking the flat width as the calculation width as in EN 1993-1-3 [20], AISI
73 S100 [21], AS/NZS 4600 [22] and GB 50017 [6], or assuming the corner region is $0.5t$ as in
74 EN 1993-1-5 [12] are deemed not accurate. Furthermore, current design methods developed
75 based on SHS and RHS with smaller corner radii may not be accurate for higher strength steel
76 SHS and RHS with significantly larger corner radii.

77 To this end, the current work aims at investigating the influence of corner radius on the local
78 buckling resistance of cold-formed SHS and RHS and assessing the applicability of current
79 design methods on the test and FE data generated based on a practical range of corner radii. A
80 series of tensile coupon tests, geometric measurements, stub column tests, 3-point and 4-point
81 bending tests are carried out on S700 and S900 SHS specimens, which are then used for
82 validating numerical models and consequently generating parametric results considering a
83 wider range of steel grades and geometric dimensions. Based on the test and FE results, the
84 current design methods for cold-formed structures in EN 1993-1-3 [20], AISI S100 [21],
85 AS/NZS 4600 [22] and GB50017 [6], and the generalized slenderness-based resistance method
86 (GSRM) [23] are assessed. A newly proposed design method, considering the influence of
87 corner radius and developed based on the framework of GSRM [23], has also been proposed
88 and validated against the obtained results.

89 **2. Experimental study**

90 **2.1 General information**

91 An experimental program investigating the compressive and bending behaviours of cold-
92 formed HSS SHS sections is described in this section. A series of tensile coupon tests,
93 geometric imperfection measurements, stub column tests and beam tests were carried out in
94 the Structures Laboratory at the University of Bath. The specimens covered two cold-formed
95 steel grades and three SHS cross-sectional sizes, including S700 80×3, S700 100×3, and S900
96 80×3.5. The SHS specimens were manufactured by cold-rolling process and the S700 and S900
97 materials have nominal proof stresses of 700 MPa and 900 MPa, respectively. Fig. 1 depicts
98 the geometric shape of the SHS, with B , H , t and r_i denoting the outer width, outer height, wall
99 thickness and inner corner radius of the cross-section, respectively.

100 **2.2 Tensile coupon tests**

101 To obtain the material properties of the investigated cold-formed SHS, a series of tensile
102 coupons were extracted from the specimens and tested according to the procedures set out in
103 EN ISO 6892-1 [24]. For each section, three flat coupons machined from the central regions of
104 3 faces (F1-F3 in Fig. 1 and Fig. 2(a)) and two corner coupons (C1 and C2 in Fig. 1 and Fig.
105 2(b)) were prepared, resulting in a total of 15 tensile coupons. A 50 kN Instron testing frame
106 was used to carry out the tensile coupon tests at a strain rate of 0.00007/s before yielding and
107 0.00024/s afterwards [24].

108 The stress-strain curves of the tested flat and corner coupons are shown in Fig. 3. The key
109 average material properties are summarised in Table 1, where E is the Young's modulus, $\sigma_{0.2}$ is

110 the 0.2% proof stress considered as the yield stress, $\sigma_{1.0}$ is the 1.0% proof stress, σ_u is the
111 ultimate tensile strength, ε_u is the strain at σ_u , ε_f is the plastic strain at fracture calculated based
112 on the elongation over the standard gauge length of $5.65\sqrt{A_c}$, where A_c is the cross-sectional
113 area of the coupon, and n and m are the Ramberg-Osgood strain hardening parameters [25]. As
114 shown in Fig. 3, all the coupons exhibited rounded stress-strain relationships without a sharply
115 defined yield point and the corner coupons displayed higher strengths and lower ductility than
116 their flat counterparts, as can be attributed to the fact that the cold forming process introduced
117 highly localised plastic strains in the corner regions. It should be noted that some of the corner
118 coupons in this study exhibited very high stiffness, which may be attributed to a small extend
119 of flattening of the curved coupons under the tensile loading. Therefore, a Young's modulus of
120 210 GPa was adopted as the averaged value for the corner coupons (Table 1). The averaged
121 tested Young's moduli were 223, 192 and 251 GPa for the S700-80×3, S700-100×3 and S900-
122 80×3.5 corner coupons, respectively.

123 **2.3 Geometric imperfection measurements**

124 The local geometric imperfection measurements were performed using a Faro 3D scanner. The
125 geometric profiles were scanned and exported as point clouds, which were then post-processed
126 using MATLAB [26]. The imperfection measurement was only conducted on the six stub
127 column specimens, and the measured values were considered to be representative for the cross-
128 sections investigated, as the local imperfection is defined as the maximum deviation of the
129 central line to the averaged edges of each faces [27]. It was unrealistic to scan the beam
130 specimens due to that, firstly, their very large sizes did not fit the scanning table, and secondly,
131 the failure regions of the beams were localised at where the bending moment was maximised

132 during the test which means that the maximum deviation may not correspond to the actual
133 location where local buckling occurred. The maximum imperfection amplitude among the four
134 faces of each section was taken as the imperfection magnitude w_0 , as reported in Table 2 for
135 the six stub columns. The maximum out-of-flatness tolerance for cold-formed steels specified
136 as $w_{EN} = 0.008B$ (B is the width of the plate, see Fig. 1) according to BS EN 10219-2 [28] is
137 also given in Table 2 for each section. As can be seen from Table 2, all the stub column
138 specimens fulfilled the BS EN 10219-2 [28] requirement, and in general a higher imperfection
139 amplitude is associated with a higher plate slenderness $c/(t\varepsilon)$, where $c = B - 2t - 2r_1$ and $\varepsilon =$
140 $\sqrt{\sigma_{0.2}/235}$, as can be seen for the S700 80×3 and S700 100×3 specimens in Table 2.

141 **2.4 Stub column tests**

142 A total of six stub column tests (2 repeated tests for each cross-section) were carried out. The
143 geometric dimensions of the stub columns are presented in Table 2, where L is the length of
144 the stub column. The nominal column lengths were chosen to be three times the section width
145 to allow the full local buckling deformation to be captured while being short enough to preclude
146 the global buckling failure mode. The stub column tests were performed using a DARTEC
147 2000 kN hydraulic testing machine, where the specimens were fixed-supported at both ends
148 except that the axial deformation at the loading end was allowed. A displacement control
149 loading method was adopted for all the stub column tests with a speed of 0.5 mm/min. During
150 the test, the average end-shortening δ was measured by two Linear Variable Displacement
151 Transformers (LVDTs) placed at two opposite corners of the specimen, the strain development
152 was monitored by four strain gauges mounted at the mid-height of the specimen and at a
153 distance equal to four times the wall thickness from the external plane of the adjacent face, as

154 illustrated in Fig. 1, and the load was read from the loading jack directly.

155 All the stub columns failed by a typical local buckling mode, as shown in Fig. 4. The load-end
156 shortening (N - δ) curves for all the stub columns are plotted in Fig. 5. The key test results are
157 reported in Table 2, including the ultimate load N_u , the end shortening δ_u at N_u , the normalised
158 compressive capacity N_u/N_y , where $N_y = A\sigma_{0.2}$, and the nondimensional plate slenderness $\bar{\lambda}_p$
159 defined according to EN 1993-1-5 [12] accounting for a width of $B-2t-2r_i$, for each specimen.

160 **2.5 3-point and 4-point bending tests**

161 Six 3-point bending tests and three 4-point bending tests giving a variant of moment gradients
162 were carried out to investigate the effects of cross-sectional slenderness and moment gradient
163 on the moment resistance of cold-formed HSS SHS beams. The measured dimensions of the
164 beams are given in Table 3, where L_0 is the clear span between the supports. The lengths of the
165 beam specimens were designed to give a variant of moment gradient while long enough to
166 preclude shear failure modes.

167 The test setups for 4-point and 3-point bending tests are depicted in Figs. 6(a) and 6(b),
168 respectively. Steel rollers were used as supports to facilitate a simply-supported boundary
169 condition. Wooden blocks, with dimensions tightly matching those of the inner faces of the
170 sections, were inserted within the SHS beams at the support and loading point locations to
171 prevent web crippling. The deflections of the beams were measured by means of both string
172 pots and LVDTs placed at mid-span for the 3-point bending tests and at mid-span and two
173 loading points for the 4-point bending tests. The end-rotations of the beams were measured by
174 two inclinometers positioned at the support locations. The strain development was monitored

175 by two pairs of strain gauges attached to the top and bottom flanges (as shown in Figs. 6(a) and
176 6(b), with the face with welds being the web to minimise its influence on the plate buckling
177 behaviour under compression) and located at the mid-span of the 4-point bending specimens
178 and at a distance of 50 mm to the mid-span of the 3-point bending specimens. The 4-point
179 loading was achieved by a spreader beam with the loading points located at the third points of
180 the specimens (Fig. 6(a)), giving a central span of 533.3 mm. All the beam tests were carried
181 out on a 2000 kN hydraulic loading machine under displacement control with a speed of 2
182 mm/min.

183 All the 3-point bending and 4-point bending specimens displayed a failure mode of local
184 buckling of the compression flange and the compressive part of the web, as shown in Fig. 7.
185 The normalised mid-span moment-rotation ($M-\theta$) and moment-curvature ($M-\kappa$) responses are
186 plotted in Fig. 8(a) and (b) for the 3-point and 4-point bending specimens, respectively. In Fig.
187 8(a), θ is the mid-span rotation of the beams under 3-point bending determined as the sum of
188 the two end rotations, and θ_{pl} is the elastic component of the rotation corresponding to the
189 plastic moment capacity M_{pl} , calculated as $\theta_{pl} = M_{pl}L_0/(EI)$. In Fig. 8(b), κ is the curvature of
190 the central span of the beams under 4-point bending derived using the vertical displacements
191 at the two loading points and the mid-span, following the procedure adopted in [29], and κ_{pl} is
192 the elastic component of curvature corresponding to M_{pl} and calculated as $\kappa_{pl} = M_{pl}/(EI)$.

193 The maximum moments M_u achieved in the tests and the corresponding rotations θ_u (or
194 curvatures κ_u) are reported in Table 3. The cross-section slenderness $c/(t\epsilon)$ and the elastic and
195 plastic moment capacities M_{el} and M_{pl} of each specimen are also given in Table 3 for

196 comparison purposes. As expected, the results follow the general trend that the higher the cross-
197 sectional slenderness, the lower the moment resistance, and M_u of all the specimens exceeded
198 M_{el} whereas only the stockiest section S900 80×3.5 reached M_{pl} . The moment-rotation curves
199 of 3-point bending samples followed a linear relationship up to the peak moment as the failure
200 was localised next to the loading point at the mid-span, whereas the moment-curvature curves
201 of 4-point bending samples all displayed a yielding slope before failure, which can be attributed
202 to the spread of yielding over the central span before failure occurred [30][31]. The effect of
203 moment gradient on the moment capacity of the cold-formed SHS beams is shown insignificant
204 and no clear trend can be observed. This observation is in line with other studies on high
205 strength steel beam tests [30][31], although theoretically specimens under 3-point bending are
206 expected to exhibit slightly higher moment resistances than under 4-point bending [30][31].

207 **3. Numerical study**

208 **3.1 Modelling assumptions**

209 Numerical models were developed using the FE package ABAQUS [32] to firstly replicate the
210 experimental results, and subsequently to generate results with wider ranges of geometric
211 dimensions and material grades. To replicate the test results, the measured geometric
212 dimensions (Tables 2 and 3) and material properties were incorporated into the FE models. The
213 stress-strain relationships of the flat and corner coupons (as reported in Fig. 3 and Table 1)
214 were employed to model the flat and corner regions of the FE models, respectively. Previous
215 investigations on cold-formed HSS material properties [8][16] showed that parts of flat
216 portions near the corners were also strengthened due to cold-forming, hence in this study the
217 FE models were assigned extended corner regions, as detailed in the next section. The measured

218 engineering stress-strain curves were translated into the true stress-log plastic strain
219 relationship before being input into the FE models.

220 The 4-node SHELL element with reduced integration (1 integration point), S4R, was used to
221 mesh the FE models. This element has been widely adopted in modelling the buckling
222 behaviour of thin-walled steel structures under various loading cases [9][11][18][23], and has
223 been shown to be able to replicate the experimental results for steel sections with width-to-
224 thickness ratios ranging from 7 – 96 [9]. The mesh size of the flat area of the flanges was $c/10$
225 in both the transverse and longitudinal directions, where $c = B - 2t - 2r_f$. The corner region was
226 discretised by 5 elements circumferentially and adopted a mesh size of $c/10$ longitudinally to
227 be consistent with the mesh of the flat area. The boundary conditions in the stub column tests
228 were modelled by coupling the two end cross-sections to two reference points that were fixed
229 except the axial deformation of the one at the loading end. The pinned supports and loading
230 points of beams under 3-point and 4-point bending were modelled by coupling the cross-
231 sections at the support and loading point locations to reference points that were located at the
232 centroid of the corresponding sections. The vertical loads and reaction forces were applied
233 through these reference points. The reference points at the support locations were pinned in the
234 bending plane and fixed out-of-the bending plane. To improve the computational efficiency,
235 only half of the cross-sections was modelled. The modified Riks method [32] was adopted to
236 allow the pre- and post-buckling responses to be traced. Local imperfection defined in the shape
237 of the corresponding first eigenmode was assigned to the stub column and beam models, with
238 a range of amplitudes investigated in the next section.

239 Residual stresses were not explicitly modelled in this study. Cold-formed structures can be
240 subjected to longitudinal and transverse residual stresses, and the former has a major influence
241 on their structural performance [16]. The longitudinal bending residual stress was implicitly
242 considered through the tensile coupon tests, where the extracted and curved coupons were
243 reintroduced the bending residual stresses by being clamped to a straight profile in the loading
244 machine. Therefore, the tested stress-strain relationships effectively included the bending
245 residual stresses [29]. The membrane residual stresses of cold-formed HSS RHS and SHS were
246 measured in Ma et al. [16] and Somodi & Kovesdi [17], which reported consistently low
247 magnitudes of longitudinal membrane residual stresses, with the maximum value less than 20%
248 of the 0.2% proof stresses. These magnitudes are considered to have a negligible influence on
249 the finite element models [33], hence they were not explicitly modelled in the current study, as
250 similarly treated in previous research on cold-formed structures [11][33][34].

251 **3.2 Model validation**

252 The width of the extended corner region and the amplitude of local imperfection to be adopted
253 in the parametric study were determined through a sensitivity study, which considered a series
254 of combinations of widths of the extended corner region ($b_c = 0.5t, t$ and $2t$) and local
255 imperfection amplitudes ($w = c/50, c/100, c/200, c/300$ and w_0). In the model validation, a total
256 of 180 models were created. Table 3 summarises the mean and coefficient of variation (COV)
257 of the ratios of FE-to-test ultimate loads ($N_{u,FE}/N_{u,TEST}$ and $M_{u,FE}/M_{u,TEST}$) achieved in the stub
258 column tests and beam tests for each combination of w and b_c . It shows that that the
259 combination of $w = c/100$ and $b_c = 2t$ gives the best estimation of the test results with mean
260 $N_{u,FE}/N_{u,TEST}$ and $M_{u,FE}/M_{u,TEST}$ ratios of 1.00 and COV of 0.036 and 0.006, and was therefore

261 employed in the parametric study. The extended corner region of $2t$ is in line with previous
262 research on cold-formed HSS sections [9][18][33]. The imperfection amplitude of $c/100$ is of
263 a similar magnitude as the BS EN 10219-2 [28] requirement $0.008B$ (approximately $c/125$)
264 while being slightly on the conservative side. Typical comparisons of the test and FE load-
265 displacement curves of stub columns and moment-rotation (or curvature) curves of beams
266 under 3-point (or 4-point) bending are given in Figs. 9 and 10, respectively. Typical stub
267 column and beam failure modes of the FE models are also compared with the corresponding
268 experimental modes in Figs. 4 and 7, respectively. These numerical and graphical comparisons
269 confirm that the developed FE models can accurately replicate the experimental results and can
270 be used in the subsequent parametric study.

271 **3.3 Parametric study**

272 Following the validation of the FE models, a parametric study was carried out to expand the
273 current data pool and to investigate the effect of corner radius as a new dimensional constraint
274 found for cold-formed HSS sections. The parametric study covered four steel grades (S500,
275 S700, S900 and S1100), twenty cross-section slendernesses ranging from Class 1 to Class 4
276 according to the Eurocode 3 definition [3], and a practical range of inner corner radii (r_i) as
277 required for cold-forming different steel grades, including $r_i = 0.5t, t, 1.5t$ and $2t$ for S500, $r_i =$
278 $0.5t, t, 1.5t, 2t$ and $2.5t$ for S700, and $r_i = t, 1.5t, 2t, 2.5t$ and $3t$ for S900 and S1100, respectively.
279 The input material parameters in the S700 and S900 FE models were defined based on the
280 average coupon test results in Section 2.2 and those in the S500 and S1100 FE models were
281 obtained from previous studies [16][17]. The stress-strain curves for the flat and corner regions
282 for all the steel grades are given in Fig. 11 with the key material properties summarised in Table

283 1 and Table 6. To eliminate the strength enhancement effect from element interaction, the stub
 284 column models employed SHS and the beam models were defined with a depth-to-width ratio
 285 of $\sqrt{k_w/k_f} = 2.44$ to give similar plate slenderness $\bar{\lambda}_p$ of the flange and web, where $\bar{\lambda}_p$ is
 286 defined in Eqs. (1) and (2), σ_{cr} is the elastic buckling load of the plate, k is the elastic buckling
 287 coefficient with $k_w = 23.4$ and $k_f = 4$ for the web (i.e. stiffened plate under bending) and flange
 288 (i.e. stiffened plate under compression), respectively, and $c = B-t$ for the calculation considered
 289 herein. The stub column models covered three common widths of 100, 200 and 300 mm and
 290 the beam models covered two widths of 100 mm and 200 mm. In total, 1140 stub column
 291 models and 1594 beam models were analysed.

$$\bar{\lambda}_p = \sqrt{\sigma_{0.2} / \sigma_{cr}} \quad (1)$$

$$\sigma_{cr} = \frac{kE\pi^2 t^2}{12(1-\nu^2)c^2} \quad (2)$$

292 3.4 Effect of corner radius

293 The results from the parametric study show that the corner radius has different influences on
 294 the buckling resistances of cold-formed HSS cross-sections in different slenderness ranges. To
 295 illustrate this phenomenon, Figs. 12(a) and (b) plot the normalised resistances of S700 SHS
 296 100×100×3 stub columns and S500 RHS 80×195×3 beams under 3-point bending with
 297 different cross-sectional slendernesses $\bar{\lambda}_p$, respectively. The horizontal axis in Figs. 12(a) and
 298 (b) is r_i/t , and the vertical axis is the obtained compressive capacity, N_u , (or moment capacities,
 299 M_u) normalised by the yielding load, N_y , (or elastic moment, M_{el}) of the cross-section, which
 300 is further normalised by the case of N_u/N_y (or M_u/M_{el}) at $r_i/t = 1$, to facilitate a straightforward
 301 comparison. Increasing the corner radius in SHS and RHS can reduce both the buckling
 302 resistance [35] and the cross-sectional area (hence the yield load). The latter is more

303 pronounced than the former in relatively stocky sections (i.e. $\bar{\lambda}_p < 1.0$), thereby leading to
304 significantly increased normalised resistances, as shown in Figs. 12(a) and (b) for both the
305 compressive and bending loading cases. For more slender cross-sections (i.e. $\bar{\lambda}_p > 1.0$), the
306 reduction in the cross-section area by increasing r_i is minimal due to the small wall thicknesses
307 whereas the reduction in the buckling resistance is more phenomenal, resulting in reduced
308 normalised resistances as shown in Figs. 12(a) and (b). These need to be considered in the
309 development of design rules for cold-formed HSS sections, as HSS require higher corner radii
310 during cold-forming than normal strength steels.

311 **4 Design methods**

312 In this section, the obtained test and numerical results were used to assess the suitability of the
313 design provisions for cold-form structural steel members in Europe (EN 1993-1-3 [20]), North
314 America (AISI S100 [21]), Australia (AS/NZS 4600 [22]) and China (GB 50017 [6]) and the
315 generalized slenderness-based resistance method (GSRM) [23]. It should be noted that there
316 are other design codes that apply to general steel structures, such as EN 1993-1-1 [3], AISC
317 360 [4], AS 4100 [5], however, in this paper, only those developed specifically for cold-formed
318 structures are examined. Firstly, the slenderness limits distinguishing compact, semi-compact
319 and slender cross-sections in the current design provisions were examined. Then the design
320 methods in these provisions for calculating the compression and bending resistances of slender
321 (i.e. Class 4, [20]) cross-sections were evaluated, including the effective width method (EWM,
322 as adopted in EN 1993-1-3 [20], AISI S100 [21], AS/NZS 4600 [22] and GB 50017 [6]), the
323 direct strength method, (DSM [13][15], as adopted in AISI S100 [21]) and GSRM [23].
324 Furthermore, a new design equation for steel grades S900 and above was proposed and

325 examined against the test and FE results.

326 The suitability of these design rules was assessed based on reliability analyses according to the
327 Annex D of EN 1990 [36]. While detailed reliability analysis procedures may be referred to
328 previous works by the authors [9][11][37], the adopted material and geometric variation
329 parameters are given herein, and the key results as indicated by the partial safety factor γ_{M0} are
330 summarised. The mean to nominal yield strength ratios (i.e. material overstrength) for S500,
331 S700, S900 and S1100 specimens adopted 1.17, 1.15, 1.16 and 0.99, with the coefficient of
332 variation of 0.057, 0.070, 0.083 and 0.072, respectively, as recommended in [14]. The
333 coefficient of variation of the geometric properties V_g was taken as 0.02 [38]. AISI S100 [21]
334 and AS/NZS 4600 [22] adopt the same safety factors for cold-formed steel design and the
335 equivalent partial safety factors γ_{M0} for compression members and flexural members are 1.05
336 and 1.18, respectively. In EN 1993-1-3 [20] and GB 50017 [6], the compression and bending
337 members are designed employing one safety factor, which is equivalent to $\gamma_{M0} = 1.0$ and 1.12,
338 respectively. Therefore, the design rules were assessed against their respective target safety
339 factors.

340 **4.1 Slenderness limits**

341 The cross-sectional classification is a key approach in the design of structural steel sections
342 with plated elements. In general, plated cross-sections are classified into compact, semi-
343 compact and slender sections (as in AISI S100 [21], AS/NZS 4600 [22]), which respectively
344 correspond to the Class 2, 3 and 4 sections according to the Eurocode 3 [3] definition. The
345 compact sections (Class 2) are those able to achieve full plastic moments under bending, the

346 semi-compact sections (Class 3) sections are those that can reach their elastic moments under
347 bending and full cross-section yielding under compression, and the slender (Class 4) sections
348 cannot attain full cross-sectional yielding under compression and elastic moments under
349 bending due to premature local buckling of the plated elements under compression.

350 The slenderness limits distinguishing these three types of cross-sections for internal (stiffened)
351 elements (as appearing in SHS and RHS) in EN 1993-1-3 [20], AISI S100 [21], AS/NZS 4600
352 [22], GB 50017 [6] and GSRM [23] are summarised in Table 7, which are expressed in terms
353 of $c/(t\varepsilon)$, where ε is $\sqrt{\sigma_{0.2}/235}$ and c is the plate width under consideration. It should be
354 noted that the plate width c is defined differently in different design codes, as detailed in Table
355 7. The translation of AISI S100 and AS/NZS 4600 limits (in terms of $\bar{\lambda}_p$) to the $c/(t\varepsilon)$ format
356 adopted the assumption of $E = 210000 \text{ N/mm}^2$. The slenderness limits in EN 1993-1-3 [20], A
357 AISI S100 [21], AS/NZS 4600 [22] and GB 50017 [6] were originally developed based on
358 normal strength carbon steels, and those in GSRM [23] were based on hot-finished normal
359 strength steels and high strength steels, therefore their validity for cold-formed high strength
360 steels requires a careful assessment.

361 While the yield limits (Class 3 limits) for internal elements under bending can be only assessed
362 by the beam results, those for internal elements under compression can be assessed by both the
363 stub column and beam results, as the four flanges of the stub columns and the upper flange of
364 the beams are under uniform compression. Figs. 13(a) and (b) plot the normalised section
365 capacities (i.e. N_u/N_y for stub columns and M_u/M_{el} for beams) varying against the $c/(t\varepsilon)$ of the
366 flange and web, respectively. The corresponding yield limits in the current design provisions

367 are also given in Figs. 13(a) and (b). It can be seen from Fig. 13(a) that the beam data gives
368 significantly more conservative results than the stub column data, therefore the reliability
369 analyses of the yield limits for internal elements under compression were based on the stub
370 column results only. The unfactored comparisons in Figs. 13(a) and (b) reveal that in general,
371 the GSRM [23] limits are the most accurate yet conservative for both cases. The obtained
372 partial safety factors γ_{M0} are summarised in Tables 8 and compared with the respective target
373 values as given in different design guidance. Table 8 shows that the target partial safety factors
374 of the codified yield limits are generally violated, except for the AISI S100 (AS/NZS 4600)
375 and GSRM ones for internal elements under bending. This suggests that the derived partial
376 safety factors for each steel grade as reported in Table 8 may be adopted for the design of cold-
377 formed HSS SHS and RHS.

378 The plastic limits (Class 2) can be only assessed by the bending results, as shown in Figs. 14(a)
379 and (b) for internal elements under compression (flange) and bending (web), respectively. The
380 unfactored comparisons in Fig. 14 indicate that the GSRM limits are the most suitable ones
381 among others. The calculated partial factors γ_{M0} for the plastic limits are summarised in Table
382 9. It can be seen that the GSRM limits require the smallest γ_{M0} (ranging 0.92~1.54) among
383 others and hence are the most suitable. Nevertheless, the derived γ_{M0} values based on the test
384 and FE results are greater than the respective codified γ_{M0} values, apart from the GSRM limit
385 for S500 internal element under bending. Moreover, the derived γ_{M0} value is shown to increase
386 with the steel grade, revealing that the current codified plastic limits for cold-formed HSS SHS
387 and RHS become less suitable as the steel grade gets higher. Overall, it is suggested herein that
388 the GSRM yield and plastic limits should be adopted for the design of cold-formed HSS SHS

389 and RHS in conjunction with the recommended safety factors of 1.0 and those reported in
390 Tables 8 and 9 if higher than 1.0.

391 **4.2 Design of slender (Class 4) cross-sections**

392 Currently, there are mainly three methods in calculating the local buckling resistances of
393 slender (Class 4) cross-sections, including the widely adopted Effective Width Method (EWM)
394 (adopted in Eurocode 3 [20], AISI S100 [21], AS/NZS 4600 [22], GB 50017 [6]), the Direct
395 Strength Method (adopted in AISI S100 [21], AS/NZS 4600 [22]), and the newly proposed
396 generalised slenderness-based resistance method (GSRM) [23]. In this section, the applicability
397 of these methods to cold-formed HSS SHS and RHS considering a practical range of corner
398 radii and steel grades is assessed based on the test and FE data from the current study.

399 **4.2.1 Plate slenderness of SHS and RHS**

400 As detailed in Table 7, the calculation of the plate slenderness (i.e. in terms of $c/(t\epsilon)$ or $\bar{\lambda}_p$) of
401 SHS and RHS in different design provision adopts different plate width c : EN 1993-1-1 [3],
402 AISI S100 [21] and AS/NZS 4600 [22] use the flat width $c = B - 2t - 2r_i$; GB 50017 [6] adopts c
403 $= B - 2t$; EN 1993-1-5 [12] recommends to adopt $c = B - 3t$ for RHS members; and EN 1993-1-3
404 [20] used the distance between the centre points of two adjacent corners and this gives $c = B -$
405 $2t - 2r_i + \sqrt{2r_i}$ for RHS. GSRM [23] and DSM [13][15] consider the local buckling of the cross-
406 section as a whole rather than its individual plate element, and suggest to use the elastic critical
407 buckling stress, σ_{cr} , of the whole cross-section to calculate the plate slenderness $\bar{\lambda}_p =$
408 $\sqrt{\sigma_{0.2}/\sigma_{cr}}$. The software CUFSM [13][15] developed based on constrained and unconstrained
409 finite strip method is commonly used to estimate the elastic critical buckling stress σ_{cr} of thin-

410 walled cross-sections. Additionally, it was found in [33] that the central line width $c = B-t$ gives
411 the closest estimation of the plate slenderness of SHS as employing the CUFSM. Therefore,
412 different ways of calculating $\bar{\lambda}_p$ are assessed herein using the CUFSM calculated value
413 $\bar{\lambda}_{\text{CUFSM}}$ as the reference. In Fig. 15, the plate slenderness $\bar{\lambda}_{p1}$, $\bar{\lambda}_{p2}$, $\bar{\lambda}_{p3}$ and $\bar{\lambda}_{p4}$ calculated
414 based on $c = B-2t-2r_i+\sqrt{2r_i}$, $B-2t-2r_i$, $B-3t$, and $B-t$, respectively, for all the cross-sections
415 modelled in the parametric study are compared to their corresponding $\bar{\lambda}_{\text{CUFSM}}$ values. Fig. 15
416 shows that the central line width $c = B-t$ gives the closest estimation of $\bar{\lambda}_{\text{CUFSM}}$ with a mean of
417 $\bar{\lambda}_{p4}/\bar{\lambda}_{\text{CUFSM}} = 0.973$ and $\text{COV} = 0.015$, which is, however, slightly unconservative.

418 To this end, the current study proposes an alternative hand-calculation method for obtaining
419 accurate and safe $\bar{\lambda}_p$ values for SHS (or RHS) for practical use when CUFSM is not available.
420 Zeinoddini and Schafer [35] reported a study on the effect of large corner radius on the plate
421 buckling strength of cold-formed (or cold-bended) structures, which suggested the adoption of
422 a reduced buckling coefficient, k_{reduced} , as shown in Eq. (3), to calculate the elastic critical
423 buckling stress of plates supported by rounded corners by replacing k in Eq. (2). It should be
424 noted that in [35] k_{reduced} is applied to $c = B-2t-2r_i$ (i.e. the flat width of the plate). However,
425 k_{reduced} from Eq. (3) does not apply for those with $r_i/t < 4$, whereas the results in Fig. 12 indicate
426 the need of taking the effect of corner radius into account for cross-sections with smaller r_i/t
427 ratios. Therefore, the current study proposes a modification of Eq. (3) to Eq. (4), with the
428 generated k_{reduced} to be used in conjunction with Eq. (5) to calculate the modified plate
429 slenderness, $\bar{\lambda}_{\text{pm}}$. It should be noted that Eq. (4) was proposed based on back calculating the
430 $\bar{\lambda}_{\text{CUFSM}}$ values and assuming a central-line width of $c = B-t$. The $\bar{\lambda}_{\text{pm}}$ values are also plotted in
431 Fig. 15, closely in line with the $\bar{\lambda}_{\text{CUFSM}}$ values with a mean $\bar{\lambda}_{\text{pm}}/\bar{\lambda}_{\text{CUFSM}} = 0.998$ and a $\text{COV} =$

432 0.019. It is suggested that the proposed $\bar{\lambda}_{pm}$ may be adopted for calculating the plate
 433 slenderness of thin-walled SHS and RHS when computational tools are not available.

$$k_{\text{reduced}} = (1.08 - 0.02 \cdot \frac{r_i}{t})^2 \cdot k \quad (3)$$

$$k_{\text{reduced}} = (1.01 - 0.02 \cdot \frac{r_i}{t})^2 \cdot k \quad (4)$$

$$\bar{\lambda}_{pm} = \sqrt{\frac{\sigma_{0.2}}{\sigma_{cr,m}}} = \frac{B - t}{t} \sqrt{\frac{\sigma_{0.2} 12(1 - \nu^2)}{k_{\text{reduced}} E \pi^2}} \quad (5)$$

434 4.2.2 Assessment of current design methods for slender cross-sections

435 The Effective Width Method (EWM) is adopted in EN 1993-1-5 [12], AISI S100 [21], AS/NZS
 436 4600 [22] and GB 50017 [6] for predicting the resistance of plated elements by assuming a part
 437 of the plate (i.e. an effective area) achieves full yielding while the remaining area resists zero
 438 force. The reduction of the whole area to the effective area, ρ , of a steel plate depends on its
 439 boundary condition, stress distribution and slenderness. For internal (stiffened) plates, ρ is
 440 expressed as Eq. (6) [12], where ψ is the stress distribution factor, with $\psi = 1$ for elements
 441 under uniform compression and $\psi = -1$ for those under bending.

$$\rho = \begin{cases} 1.0 & \text{for } \bar{\lambda}_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \frac{\bar{\lambda}_p - 0.055(3 + \psi)}{\lambda_p^2} \leq 1.0 & \text{for } \bar{\lambda}_p > 0.5 + \sqrt{0.085 - 0.055\psi} \end{cases} \quad (6)$$

442 The Direct Strength Method (DSM) was proposed by Schafer and Peköz [13][15] for
 443 estimating the buckling resistance of cold-formed steel members taking into account the
 444 combined effects of local and global instabilities simultaneously and has been incorporated in
 445 the AISI S100 [21] and AS/NZS 4600 [22]. The design equation of DSM is generalised to apply
 446 to both compression and bending loading cases, as given in Eq. (7), where $\lambda_l = \sqrt{N_y/N_{cr1}}$ and
 447 $\sqrt{M_{el}/M_{cr1}}$ for cross-sections under compression and bending, being N_{cr1} and M_{cr1} the critical

448 buckling load and moment, respectively, considering both local and global buckling. Since the
 449 current study focuses only on the local buckling response, N_{cr1} and M_{cr1} only account for local
 450 plate buckling and λ_1 is equivalent to $\bar{\lambda}_p$ herein.

$$\frac{N_u}{N_y} \text{ (or } \frac{M_u}{M_{el}}) = \begin{cases} 1.0 & \text{for } \lambda_1 \leq 0.776 \\ \left[1 - 0.15 \left(\frac{1}{\lambda_1} \right)^{0.8} \right] \left(\frac{1}{\lambda_1} \right)^{0.8} & \text{for } \lambda_1 > 0.776 \end{cases} \quad (7)$$

451 The generalized slenderness-based resistance method (GSRM) was developed to understand
 452 better the behaviour and strength of hollow sections and improve the economy and practicality
 453 of design rules dedicated to these types of sections [23]. The GSRM design equation [23]
 454 adopts the basic structure of Winter's formula (as used by EWM) with modified factors for
 455 slender cross-sections, as shown in Eqs. (8) and (9), where φ_1 and φ_2 indicate the loading
 456 conditions with $\varphi_1 = \varphi_2 = 1$ for hollow sections under compression and $\varphi_1 = 1$ and $\varphi_2 = -1$ for
 457 those under bending, respectively.

$$\frac{N_u}{N_y} \text{ (or } \frac{M_u}{M_{el}}) = \frac{1}{\lambda_L} \left(1 - \frac{A}{\lambda_L} \right), \text{ for } \lambda_L \geq 0.5 + \sqrt{0.25 - A} \quad (8)$$

$$A = 0.225 + 0.025\psi_2 \frac{1 + \psi_1}{2} \quad (9)$$

458 The EWM [12], DSM [13][15] and GSRM [23] design curves are compared with the test and
 459 FE data in Figs. 16(a) and (b) for cross-sections under compression and bending, respectively.
 460 To avoid the inaccuracy sourced from adopting different c values, the assessment was based on
 461 $\bar{\lambda}_{CUFSM}$. It should be noted that the EWM for beam members involves iterations in the
 462 calculation process and cannot be expressed by a $\bar{\lambda}_p$ - M_u/M_{el} curve, therefore it is not included
 463 in Fig. 16(b). Careful reliability analyses of the EWM, DSM and GSRM based on the test and
 464 FE data have also been carried out. Table 10 summarises the required safety factors γ_{M0} for each

465 steel grade and for each design method, where the γ_{M0} values for EWM and GSRM are shown
466 to be smaller than or around 1.0 except those for the S1100 specimens, indicating their
467 suitability for giving safe prediction to the local buckling resistances of cold-formed HSS SHS
468 and RHS, although with some extent of conservativeness. The DSM safety factors are generally
469 larger than 1.0 indicating that higher partial safety factors should be used in conjunction with
470 the method. It should be noted that the higher partial safety factors obtained for the S1100
471 specimens are mainly attributed to the relatively material overstrength factor used [14]. It is
472 expected that the safety factors for S1100 cross-sections may be re-evaluated once more
473 material data of S1100 steel is available.

474 **4.2.3 Proposed design curve for S900 and higher steel grades**

475 It can be observed in Fig. 16 and Table 10 that the current design method GSRM [23] can
476 accurately predict the local buckling resistances of S500 and S700 cold-formed SHS and RHS
477 under compression and bending, it still underestimates the S900 and S1100 data points. To this
478 end, a new design curve is proposed specifically for the design of S900 and above cold-formed
479 RHS and SHS, as given in Eqs. (10) and (11), which take the format adopted for GSRM [23]
480 (Eqs. (8) and (9)). The constant factors in Eqs. (10) and (11) were derived by least-square fitting
481 considering all the S900 and S1100 datapoints below N_u/N_y or $M_u/M_{el} = 1$. Consequently, Eqs.
482 (10) and (11) are generalised for both loading cases of uniform compression and bending, with
483 $\varphi_1 = \varphi_2 = 1$ for compression and $\varphi_1 = 1$ and $\varphi_2 = -1$ for bending, respectively. The proposed
484 design curve is also plotted and compared with the test and FE data in Fig. 16. The required
485 safety factors of the proposed design curve for S900 and S1100 specimens are given in Table
486 10, which are closer but still less than 1.0 for the S900 specimens, indicating their suitability.

487 In addition, it is recommended to use a safety factor of 1.25 for S1100 specimen based on the
488 existing material data [16].

$$\frac{N_u}{N_y} \left(\text{or } \frac{M_u}{M_{el}} \right) = \frac{1}{\bar{\lambda}_p} \left(1 - \frac{0.16 + C}{\bar{\lambda}_p} \right), \text{ for } \bar{\lambda}_p \geq 0.5 + \sqrt{0.09 - C} \quad (10)$$

$$C = 0.02\psi_2(1 + \psi_1) \quad (11)$$

489 **5 Conclusions**

490 An experimental and numerical study on the local buckling resistances of cold-formed has been
491 carried out. The experimental work included tensile coupon tests, geometric imperfection
492 measurements, 6 stub column tests and 6 three-point bending and 3 four-point bending tests.
493 Finite Element (FE) models have been developed to replicate the experiments, which were then
494 used in subsequent parametric studies considering various steel grades, cross-section
495 dimensions and a practical range of corner radii. Based on the obtained results, the slenderness
496 limits and design rules for slender cross-sections in various design provisions were assessed by
497 means of reliability analyses. The results suggest that among all the design provisions
498 examined, the GSRM [23] is the most suitable method in terms of accuracy and safety, although
499 with some extent of conservativeness for S900 steel and above. To this end, the study proposed
500 a new local buckling design curve in the framework of GSRM [23] for the design of cold-
501 formed HSS SHS and RHS with S900 steel and above. Additionally, a new calculating method
502 for deriving the cross-sectional slenderness of RHS and SHS considering the influence of
503 corner radius has been proposed to be used when computational tools are not available.

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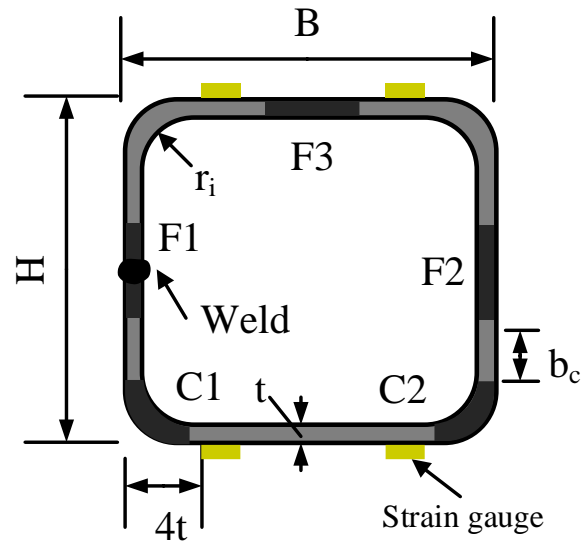
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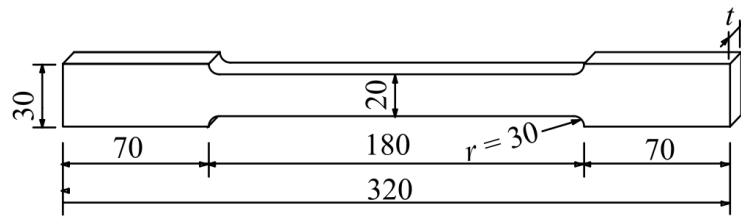
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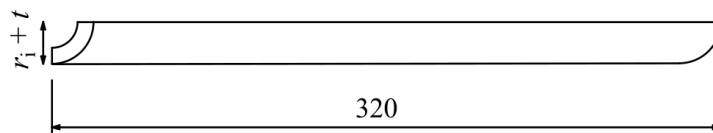
Fig.1 Notations of cross-sectional dimensions and locations of strain gauges



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(a) Flat coupon



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(b) Corner coupon

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Fig.2 Dimensions of tensile coupons (unit in mm)

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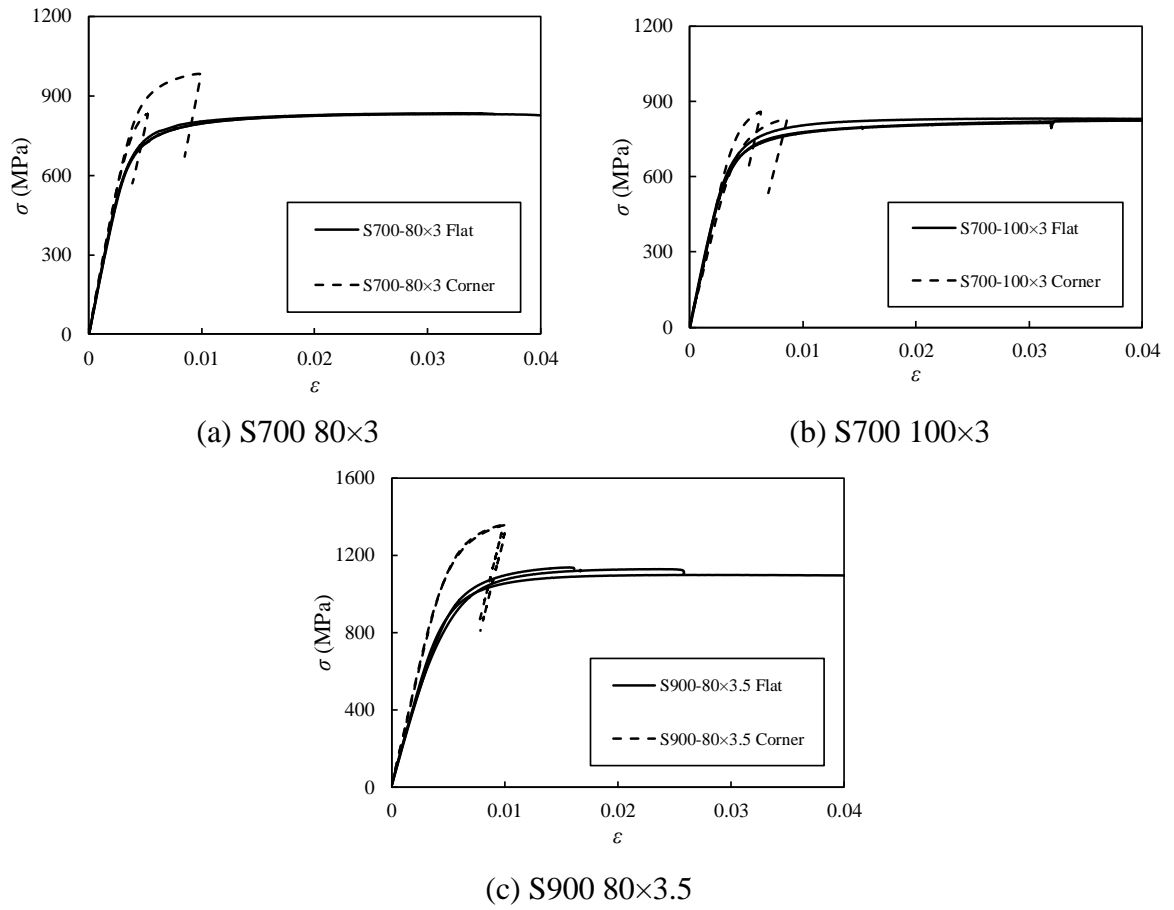


Fig.3 Measured stress-strain curves from tensile coupon tests

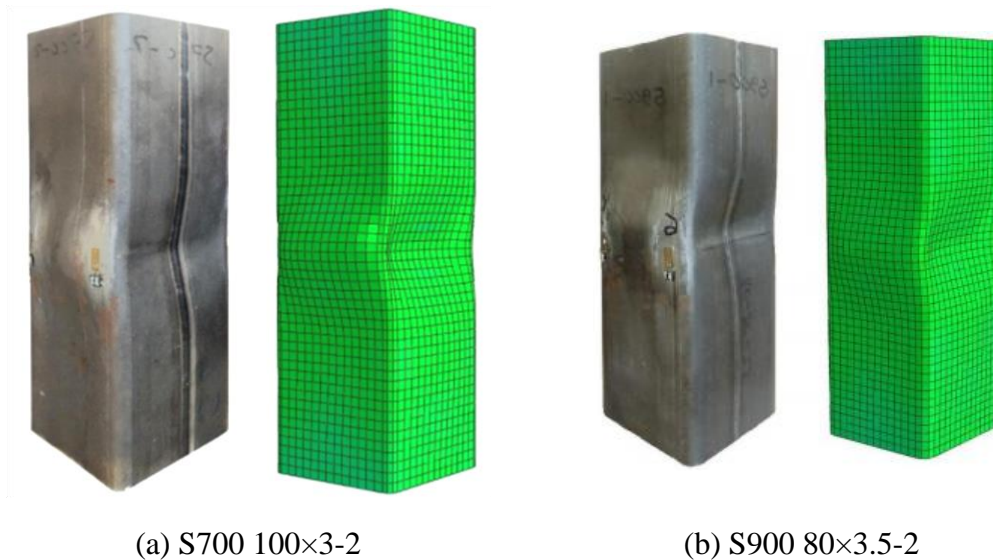


Fig. 4 Typical test and FE failure modes of cold-formed HSS SHS stub columns

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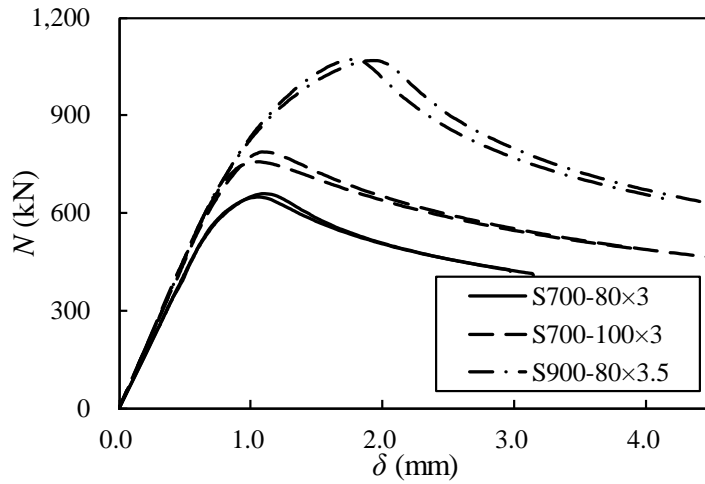
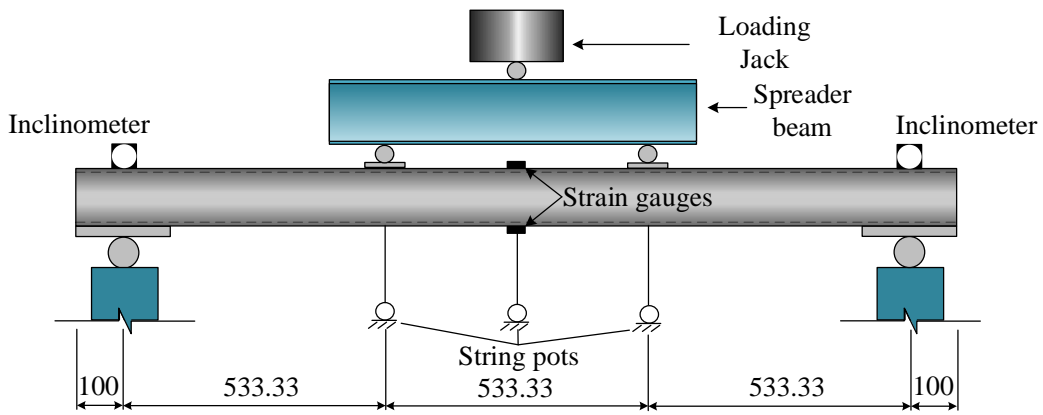
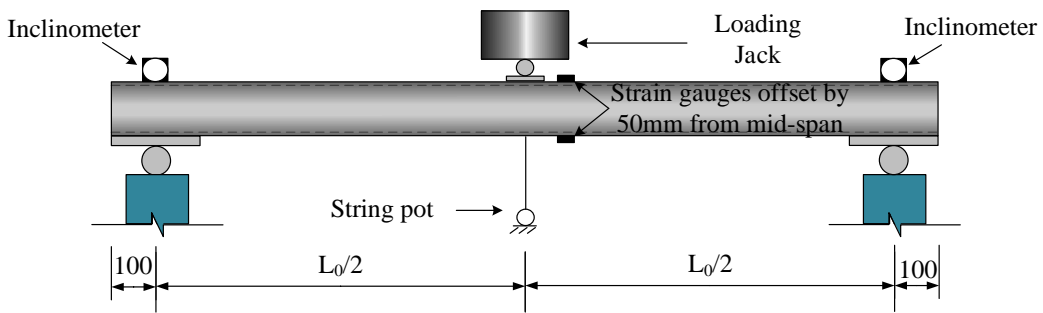


Fig. 5 Load-axial displacement curves from cold-formed HSS SHS stub column tests

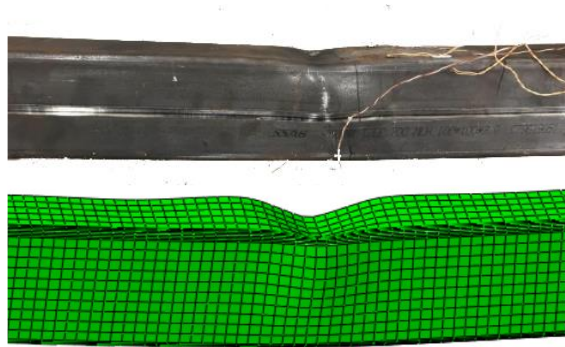


(a) 4-point bending (unit in mm)



(b) 3-point bending (unit in mm)

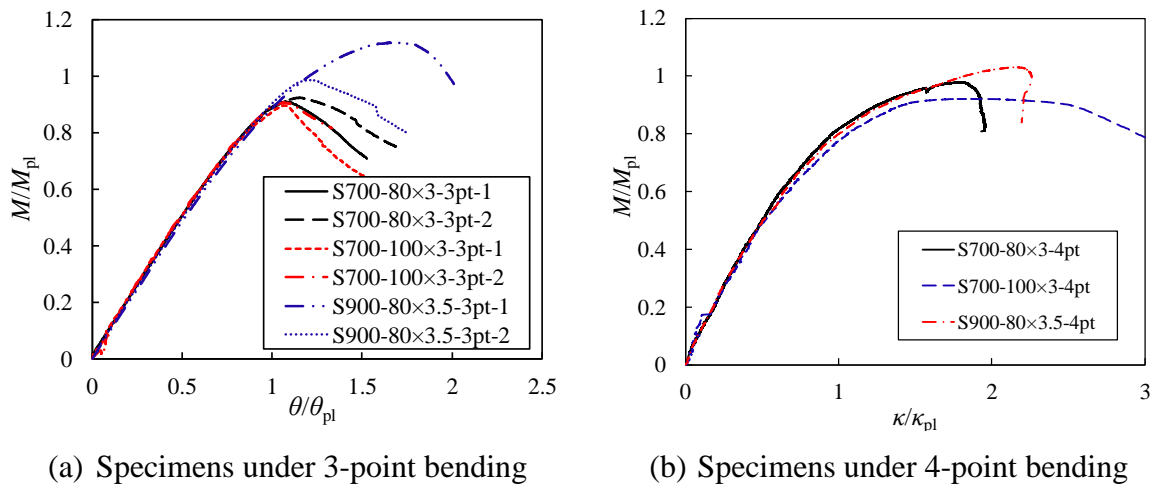
Fig. 6 Test setup of bending tests (unit in mm)



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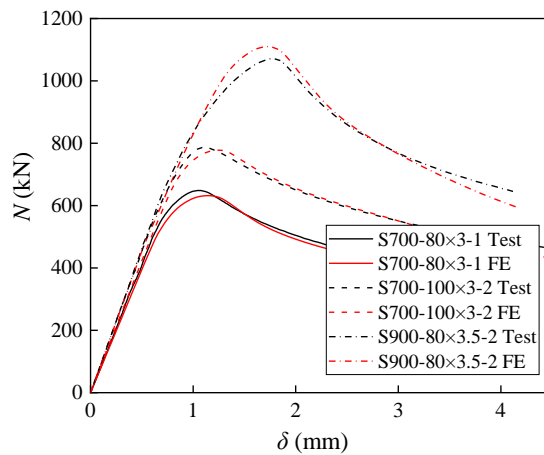
615 Fig. 7 Typical test and FE failure modes of cold-formed HSS SHS under 3-point bending

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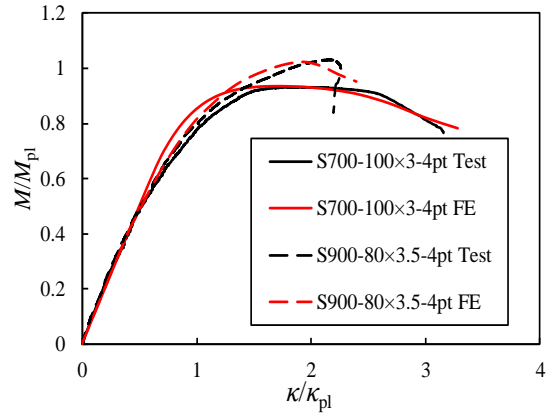
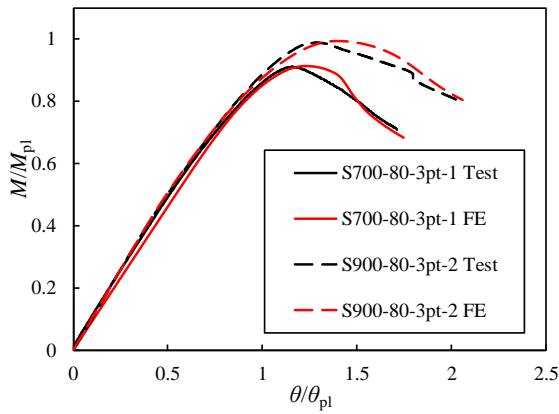
617 Fig. 8 Normalised moment-rotation (or curvature) responses of cold-formed HSS SHS beams
618 under 3-point (or 4-point) bending

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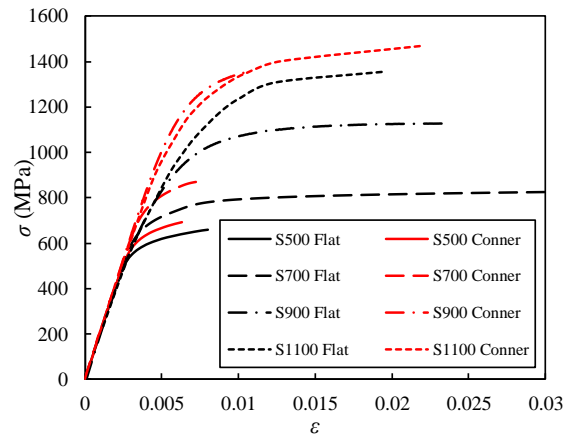
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621 Fig. 9 Comparisons of test and FE load-axial displacement curves of SHS stub columns



(a) Specimens under 3-point bending (b) Specimens under 4-point bending
 Fig. 10 Comparisons of test and FE load vs end-rotation (or curvature) relationships of cold-formed HSS SHS beams under bending

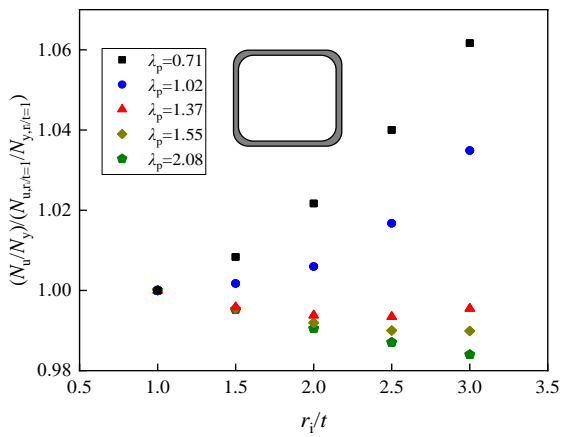
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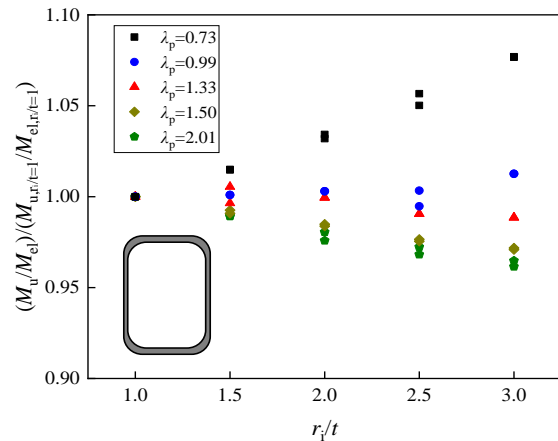
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Fig. 11 Material stress-strain relationships employed in the parametric study



(a) Under compression (S700-100x100)

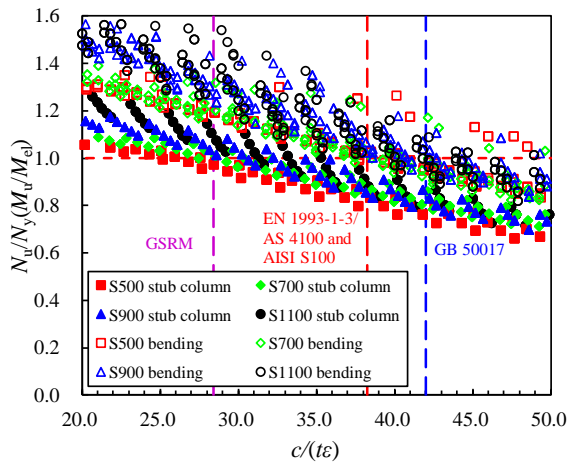


(b) Under bending (S500-80x194-3pt)

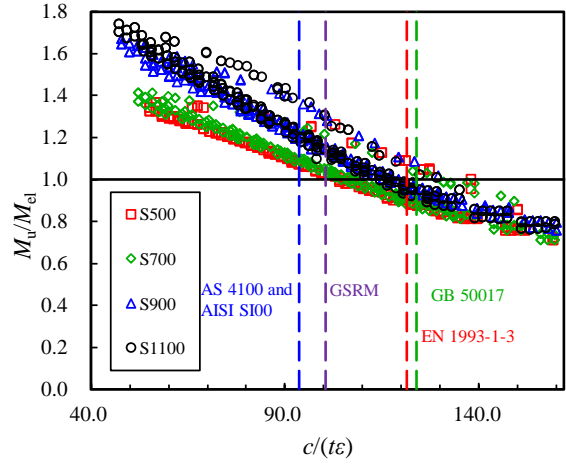
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Fig. 12 Effect of corner radius on the cross-sectional resistance of cold-formed HSS RHS



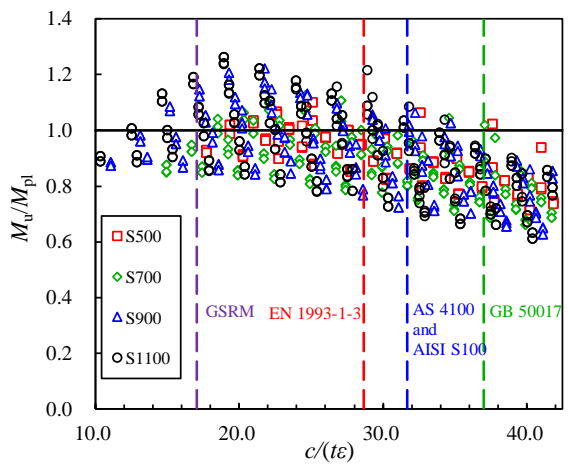
(a) Plates under compression



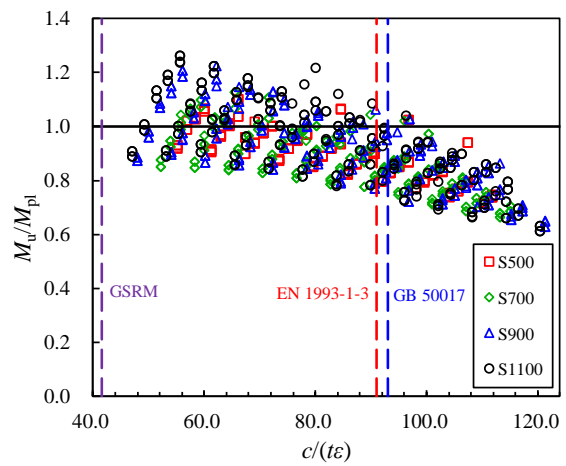
(b) Plates under bending

Fig. 13 Assessment of yield (Class 3) slenderness limits in different design provisions

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(a) Flange



(b) Web

Fig. 14 Assessment of plastic (Class 2) slenderness limits in different design provisions

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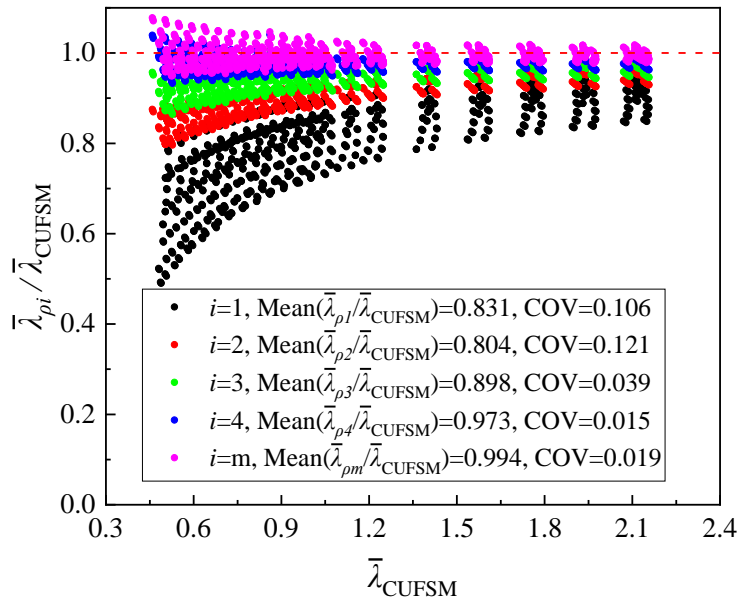


Fig. 15 Comparisons of different definitions of plate slenderness with the CUFSM calculated value

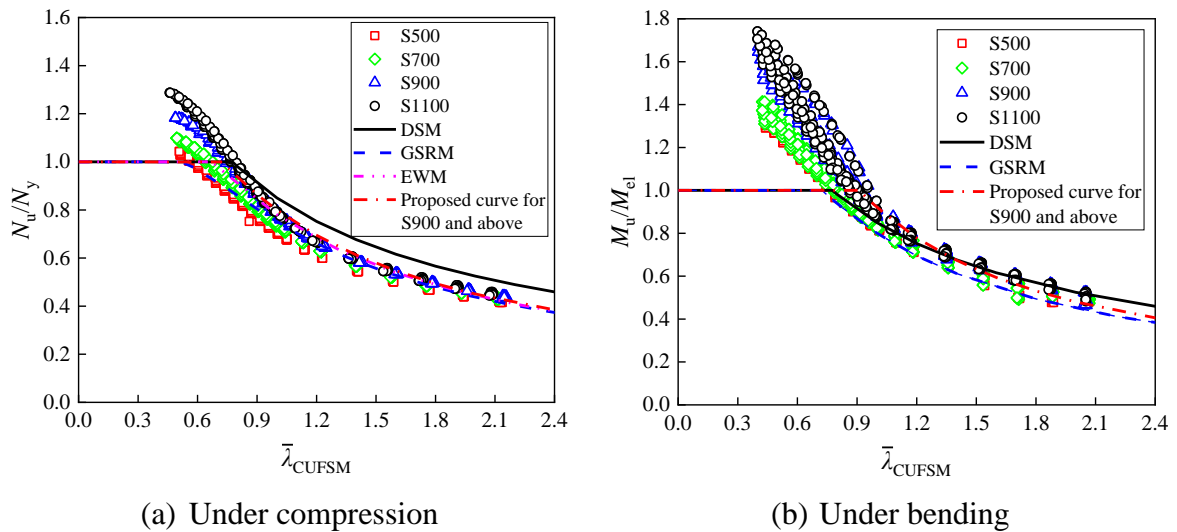


Fig. 16 Assessment of EWM [12], DSM [13][15] and GSRM [23] for the design of cold-formed HSS SHS and RHS

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635 Table 1 Average material properties obtained from tensile coupon tests

Cross-sections	Coupon	E (MPa)	$\sigma_{0.2}$ (MPa)	$\sigma_{1.0}$ (MPa)	σ_u (MPa)	ε_u (%)	ε_f (%)	m	n
S700-80×3	Flat	208800	755	828	842	2.9	12.6	10.8	4.0
	Corner	210000	926	894	985	1.0	12.2	12.2	4.1
S700-100×3	Flat	204300	731	800	827	4.0	8.5	8.5	3.9
	Corner	210000	860	805	860	0.6	13.5	13.5	4.3
S900-80×3.5	Flat	205900	986	1112	1120	2.3	6.1	6.1	3.9
	Corner	210000	1324	1323	1355	1.0	9.6	22	3.2

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637 Table 2 Measured geometric dimensions and test results of stub columns

Specimens	L (mm)	B (mm)	H (mm)	t (mm)	r_i (mm)	$\bar{\lambda}_p$	w_0 (mm)	N_u (kN)	δ_u (mm)	N_u/N_y
S700-80×3-1	238.5	80.52	80.52	3.02	3.60	0.71	0.594	648.6	1.05	0.93
S700-80×3-2	239.0	80.47	80.48	2.99	3.60	0.72	0.598	657.8	1.10	0.95
S700-100×3-1	299.5	99.96	100.61	3.25	2.80	0.86	0.567	757.9	1.05	0.84
S700-100×3-2	300.0	99.98	100.69	3.25	2.80	0.86	0.532	786.7	1.08	0.87
S900-80×3.5-1	240.0	80.69	80.81	3.52	6.00	0.66	0.315	1070.0	1.93	1.01
S900-80×3.5-2	240.0	80.62	80.75	3.55	6.00	0.66	0.338	1071.1	1.78	1.00

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639 Table 3 Measured geometric dimensions and test results of beam specimens

Specimens	L_0 (mm)	B (mm)	H (mm)	t (mm)	r_i (mm)	$c/(t\varepsilon)$	M_u (kNm)	θ_u (rad) or κ_u	M_u/M_{el}	M_u/M_{pl}
S700-80×3-3pt-1	1600	80.30	80.49	3.08	3.60	39.4	18.25	0.06	1.05	0.91
S700-80×3-3pt-2	1000	80.48	80.457	3.11	3.60	39.1	18.75	0.05	1.07	0.95
S700-100×3-3pt-1	1600	100.46	100.23	3.36	2.80	46.2	30.08	0.05	1.04	0.90
S700-100×3-3pt-2	1000	100.53	100.21	3.28	2.80	47.4	29.77	0.03	1.05	0.91
S900-80×3.5-3pt-1	1600	80.50	80.76	3.69	6.00	34.9	32.86	0.12	1.23	1.08
S900-80×3.5-3pt-2	1000	80.56	80.62	3.56	6.00	36.3	28.97	0.04	1.12	0.99
S700-80×3-4pt	1600	80.46	80.54	3.20	3.60	37.8	20.36	1.91	1.13	0.98
S700-100×3-4pt	1600	100.50	100.12	3.43	2.80	45.1	31.50	1.52	1.07	0.92
S900-80×3.5-4pt	1600	80.47	80.64	3.57	6.00	36.1	30.32	3.08	1.17	1.03

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643 Table 4 Comparison of stub column and beam test results with FE results employing varying widths
 644 of extended corner region and amplitudes of local imperfection

Specimens	$b_c = 0.5t$			$b_c = t$				$b_c = 2t$				
	$c/50$	$c/100$	$c/200$	w_0	$c/50$	$c/100$	$c/200$	w_0	$c/50$	$c/100$	$c/200$	w_0
Mean-Stub	0.92	0.98	1.03	1.01	0.92	0.98	1.03	1.01	0.94	1.00	1.05	1.03
COV-Stub	0.034	0.031	0.029	0.044	0.033	0.032	0.031	0.046	0.037	0.036	0.034	0.050
Mean-Beam	0.95	0.97	1.01	0.98	0.96	0.98	1.02	0.99	0.98	1.00	1.04	1.02
COV-Beam	0.015	0.009	0.015	0.013	0.012	0.008	0.005	0.008	0.009	0.006	0.008	0.008

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647 Table 5 Comparisons of test and FE results of stub column and beam specimens.

Specimens	$N_{u,FE}/N_{u,TEST}$	$\delta_{u,FE}/\delta_{u,TEST}, \theta_{u,FE}/\theta_{u,TEST}$ OR
	or $M_{u,FE}/M_{u,TEST}$	$\kappa_{u,FE}/\kappa_{u,TEST}$
Stub columns	S700-80×3-1	0.97
	S700-80×3-2	0.95
	S700-100×3-1	1.03
	S700-100×3-2	0.99
	S900-80×3.5-1	1.02
	S900-80×3.5-2	1.04
Beams	S700-80×3-3pt-1	1.00
	S700-80×3-3pt-2	0.98
	S700-100×3-3pt-1	1.02
	S700-100×3-3pt-2	1.01
	S900-80×3.5-3pt-1	1.01
	S900-80×3.5-3pt-2	1.00
	S700-80×3-4pt	0.97
	S700-100×3-4pt	1.00
	S900-80×3.5-4pt	0.99

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650 Table 6 The material parameters in parametric studies

Steel grade	Location	E (MPa)	$\sigma_{0.2}$ (MPa)	σ_u (MPa)	ϵ_u	Reference
S500	Flat	210000	619.0	659.0	0.0072	[18]
	Corner	210000	672.0	691.0	0.0062	
S1100	Flat	205000	1073.0	1356.0	0.0203	[16]
	Corner	206000	1245.0	1470.0	0.0221	

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653 Table 7 Slenderness limits for cold-formed SHS and RHS beams in current design provisions

Design guidance	Definition of plate width, c	Class 2 flange	Class 2 web	Class 3 flange	Class 3 web
EN 1993-1-3 [20]	$c = B - 2t - 2r_i + \sqrt{2}r_i$	28.68	91.01	38.24	121.35
AISI S100 [21]	$c = B - 2t - 2r_i$	32.4	N/A	38.24	93.64
GB 50017 [6]	$c = B - 2t$	37	93	42	124
GSRM [23]	$c = B - 2t - 2r_i$	17.04	41.65	28.4	100.52

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656 Table 8 Partial safety factors γ_{M0} for the yield (Class 3) limits of cold-formed HSS SHS and RHS

Design guidance	Under compression					Under in-plane bending				
	Target	S500	S700	S900	S1100	Target	S500	S700	S900	S1100
EN 1993-1-3 [20]	1.00	1.28	1.29	1.31	1.43	1.00	1.16	1.19	1.19	1.18
AISI S100 [21]	1.18	1.28	1.29	1.31	1.43	1.05	0.97	0.99	0.99	0.96
GB 50017 [6]	1.12	1.37	1.38	1.45	1.58	1.12	1.17	1.22	1.22	1.21
GSRM [23]	1.00	1.09	1.09	1.04	1.14	1.00	1.00	1.04	1.03	1.01

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659 Table 9 Partial safety factors γ_{M0} for the plastic (Class 2) limits of cold-formed HSS SHS and RHS

Design guidance	Target	Under compression				Under in-plane bending			
		S500	S700	S900	S1100	S500	S700	S900	S1100
EN 1993-1-3 [20]	1.00	1.25	1.39	1.39	1.72	1.16	1.46	1.45	1.69
AISI S100 [21]	1.05	1.30	1.45	1.44	1.79	N/A	N/A	N/A	N/A
GB 50017 [6]	1.12	1.38	1.52	1.52	1.89	1.18	1.47	1.46	1.76
GSRM [23]	1.00	1.10	1.24	1.23	1.53	0.92	1.19	1.18	1.44

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661 Table 10 Partial safety factors γ_{M0} for the design of cold-formed HSS SHS and RHS slender (Class 4)
662 sections

Methods	Cross-sections under compression				Cross-sections under bending			
	S500	S700	S900	S1100	S500	S700	S900	S1100
EWM [20]	0.98	0.96	0.89	1.15	0.99	0.97	1.00	1.29
DSM [13][15]	1.06	1.06	1.01	1.34	1.05	1.05	1.04	1.30
GSRM [23]	0.91	0.91	0.84	1.17	0.95	0.95	0.93	1.15
Proposed	-	-	0.91	1.23	-	-	1.00	1.24

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