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1 **Numerical Simulation and Design of Stainless Steel Hollow**
2 **Flange Beams under Shear**

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22
23 **Abstract**

24 Stainless steel offers a range of benefits over conventional carbon steel in structural
25 applications. This paper presents the detailed numerical modelling of shear response of cold-
26 formed stainless steel hollow flange sections using finite element software package, Abaqus.
27 The effect of geometric parameters such as section height and section thickness, and the
28 influence of different steel grades were investigated following the validation of finite element
29 models. From numerical results, the formation of diagonal tension fields can be clearly
30 observed in the webs of rectangular hollow flange sections while more even distribution of the
31 stresses in the webs is seen in triangular hollow flange sections. Further, a plastic hinge type
32 mechanism is formed in triangular flanges at the post-failure region. The evaluation of

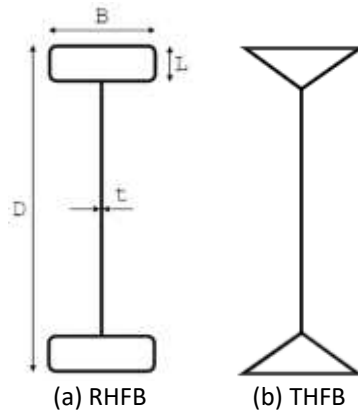
33 Eurocode 3 and the direct strength method shear design provisions for stainless steel hollow
34 flange beams are found to be significantly conservative. Therefore, modified provisions were
35 proposed and the comparison of those with finite element results confirmed the accurate and
36 consistent shear resistance predictions over the codified provisions.

37 *Keywords: Cold-formed stainless steel, Hollow flange sections, Finite element modelling,*
38 *Shear, Eurocode 3, Direct strength method*

39 **1 Introduction**

40 The increasing demand for stainless steel as a construction material can be seen over the other
41 materials in the past few decades [1]. The key feature of stainless steel is its corrosion resistance
42 making stainless steel structural components more durable while being recyclable material
43 points out stainless steel as a sustainable solution to construction wastes. Even though, stainless
44 steel costs approximately four times higher than conventional carbon steel, it is suggested in
45 studies that stainless steel structures are more economical on the basis of whole life than carbon
46 steel in aggressive conditions [2].

47 Cold-formed sections are more common among stainless steel sections compared to hot-rolled
48 and built-up sections in light structural applications [3]. There are various types of cold-formed
49 sections including open sections and hollow sections. The cross-sections of doubly symmetric
50 rectangular hollow flange beams (RHFBs) and triangular hollow flange beams (THFBs) are
51 shown in Fig. 1. These sections can be formed by connecting the cold-formed hollow flanges
52 to the web elements using electric resistance welding. The doubly symmetric hollow flange
53 sections are more stable to the torsional effects than the monosymmetric hollow flange channel
54 sections, and closed flanges suppress the distortional buckling effects which are more likely to
55 appear in open sections with free edges such as C-sections and Z-sections. Therefore, doubly
56 symmetric hollow flange sections are comparable in stability to commercially available I-
57 sections and are found to be structurally efficient than conventional cold-formed sections.



58

59 Fig. 1 Doubly symmetric hollow flange sections

60 A number of researches have investigated the structural behaviour of hollow flange sections in
 61 the past. Keerthan and Mahendran [4], [5] conducted experimental studies and numerical
 62 investigations on the shear behaviour of cold-formed steel rectangular hollow flange channel
 63 beams known as LiteSteel beams. Keerthan et al. [6], [7] investigated the combined bending
 64 and shear response of rectangular hollow flange channel sections using experimental and
 65 numerical studies. Moreover, both bending tests and numerical investigations have been
 66 conducted on the rivet-fastened rectangular hollow flange channel beams by Siahaan et al. [8],
 67 [9] while Wanniarachchi and Mahendran [10] experimented screw-fastened RHFBs to find out
 68 section moment capacities. Also, the structural behaviour of cold-formed channel sections has
 69 been thoroughly investigated by many researchers. Both experimental and numerical
 70 investigations on cold-formed steel channel sections have been conducted by Pham and
 71 Hancock [11]–[13] to study the combined bending and shear behaviour. The shear response of
 72 lipped channel sections has been studied by Keerthan and Mahendran [14] for cold-formed
 73 steel and Dissanayake et al. [15] for cold-formed stainless steel. In addition, the structural
 74 response of I-sections has been investigated by a number of studies over the years. Olsson [16]
 75 and Real et al. [17] performed shear tests on stainless steel plate girders while the bending and
 76 shear interaction behaviour of stainless steel plate girders has been investigated by Saliba and
 77 Gardner [18]. Further, the numerical investigations on lateral-torsional buckling behaviour of
 78 stainless steel I-sections have been carried out by Saadat and Ashraf [19]. However, research
 79 into cold-formed stainless steel hollow flange sections are relatively scarce.

80 The attention has been also given to the elastic shear buckling response of cold-formed sections
 81 by a number of researches [20]–[22]. Keerthan and Mahendran [22] conducted shear buckling
 82 analyses of different cold-formed sections including open and hollow flange beams using

83 numerical modelling. They proposed a generalised equation to calculate the shear buckling
84 coefficients of cold-formed sections. The proposed equation takes into account the level of
85 fixity of the web-to-flange juncture. It was suggested from the findings that the level of fixity
86 at the web-to-flange juncture of RHFBS and THFBs is closer to fixed support conditions by
87 Keerthan and Mahendran [22].

88 The direct strength method (DSM) has been adopted in the current North American
89 specifications, AISI S100 [23] and Australian and New Zealand standards, AS/NZS 4600 [24]
90 for the design of cold-formed steel sections. The DSM considers the whole section buckling
91 when determining the section resistance, therefore, takes into account the element interaction
92 in the design calculations. However, the current European standards for cold-formed steel,
93 EN19931-3 [25] and for stainless steel, EN1993-1-4 [26] do not take into account the beneficial
94 element interaction that present at the web-to-flange juncture [27]. Therefore, it is expected to
95 provide conservative resistance predictions from European standards for hollow flange
96 sections.

97 In this paper, the shear response of cold-formed stainless steel hollow flange sections (RHFBS
98 and THFBs) is discussed. The details of numerical modelling conducted to investigate the shear
99 response of RHFBS and THFBs and the use of numerical results in the evaluation of codified
100 design provisions are presented.

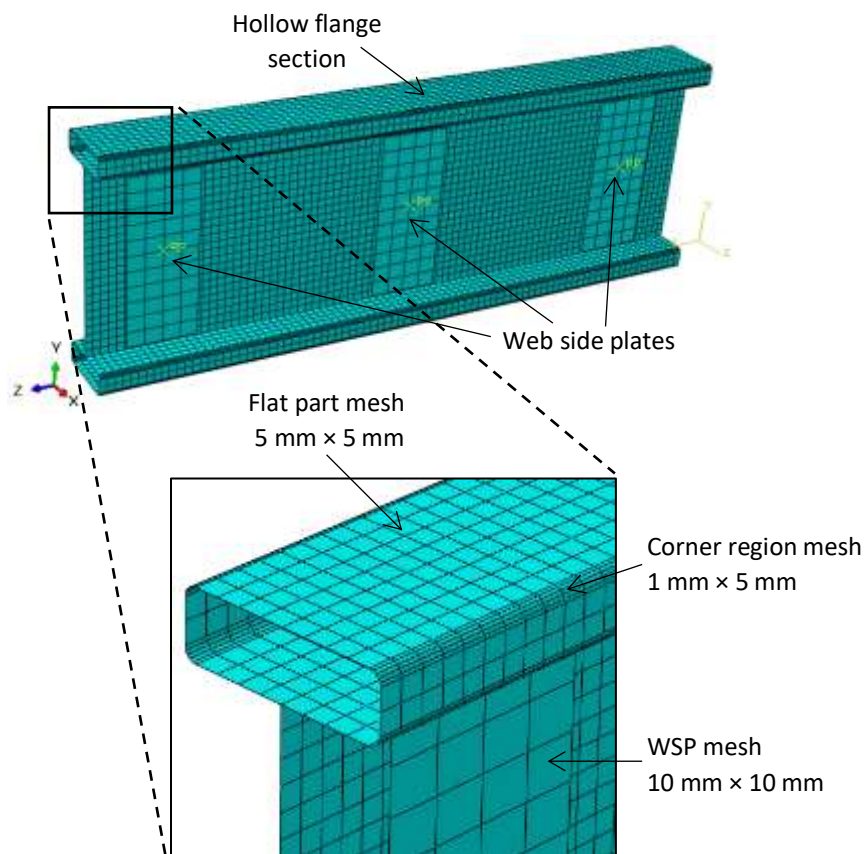
101 **2 Finite element (FE) modelling**

102 The numerical studies were conducted using commercially available FE software package
103 ABAQUS CAE 2017 to investigate the shear response of cold-formed stainless steel hollow
104 flange sections. The three-point loading setup used by Keerthan and Mahendran [4] in the shear
105 tests of single LiteSteel beams were incorporated in the development of FE models. The details
106 of numerical modelling and model validation are given in this section.

107 **2.1 Development of FE model**

108 In each FE model, single hollow flange sections were modelled together with three web side
109 plates (WSPs) placed at the supports and at the loading point to simulate three-point loading
110 tests. The quadrilateral four-node shell element with reduced integration, S4R was picked from
111 the element library for the modelling of hollow flange sections. A 5 mm × 5 mm mesh was
112 assigned for the flat parts of the sections while employing a relatively finer mesh of 1 mm × 5

113 mm to the corner regions following the mesh sensitivity analyses. The rigid quadrilateral
114 element with four nodes, R3D4 was chosen to simulate the WSPs which have a relatively
115 higher stiffness. The centre point of each plate was assigned as the rigid body reference point
116 to which the motion of the rigid plates was then coupled. A 10 mm × 10 mm mesh was assigned
117 to WSPs. Fig. 2 shows the different parts of the FE model and FE mesh.



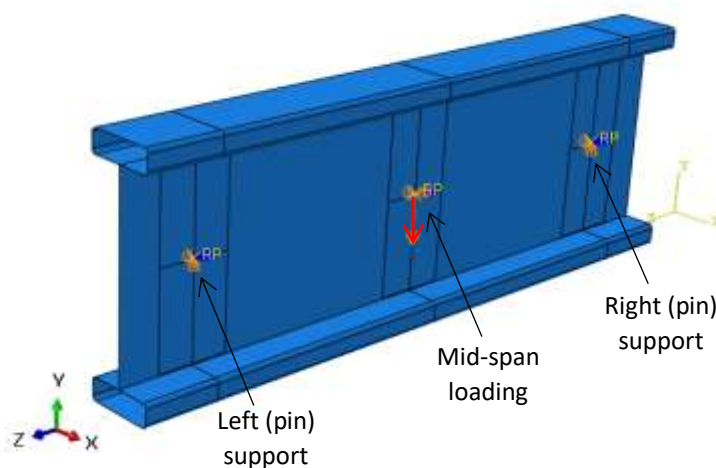
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119 Fig. 2 Assembly of parts and FE mesh used in the modelling

120 In this study, recent proposals suggested by Arrayago et al. [28] to two-stage Ramberg-Osgood
121 material model were incorporated to represent the non-linear material response of stainless
122 steel while an elastic, perfectly-plastic material model was employed to model carbon steel
123 behaviour in FE models. Then, stress-strain material data was fed into Abaqus in the form of
124 true stress (σ_{true}) and log plastic strain (ϵ_{ln}^{pl}). As a result of cold-work of forming, material
125 properties of corner regions of stainless steel cross-sections are enhanced. A number of studies
126 have investigated these strength enhancements and predictive models have been proposed
127 [29]–[31]. These induced strengths in corner regions were explicitly considered in the
128 numerical modelling and the more details of this can be found in [15]. The effects of residual

129 stresses were not incorporated in the numerical modelling of this study and were found to be
130 negligible from similar numerical studies [5], [32], [33].

131 In the three-point loading tests, WSPs were attached to the section webs to eliminate any
132 bearing failure that could occur at the supports or at the loading point. Therefore, in the FE
133 models, boundary conditions and loading were assigned to the WSPs through the coupled rigid
134 body reference points. Pin support conditions were employed at the two beam ends to maintain
135 simply supported conditions. The in-plane translational DOFs of the cross-sectional plane (x-
136 y plane) were restrained for the application of pin supports to the beam sections and the
137 rotational DOF about the longitudinal axis (z-axis) of the section was restrained to avoid
138 possible torsional effects. At the mid-span WSP, a downward displacement was applied to the
139 reference point to simulate the loading of the section. The tie constraints available in Abaqus
140 were employed to represent the bolted connections between section webs and WSPs. Fig. 3
141 illustrates the assigned boundary conditions in the FE modelling.



142

143 Fig. 3 Assigned boundary conditions in the FE modelling

144 The effects of the local geometric imperfections on the performance of thin steel section
145 behaviour is required to be taken into account in the numerical analysis. The details of
146 numerical modelling of geometric imperfections have been reviewed in previous studies [34]–
147 [36]. To calculate the magnitude of the local geometric imperfections (ω_0) of steel sections,
148 Gardner and Nethercot [34] proposed modifications to the original prediction model developed
149 by Dawson and Walker [37]. This modified Dawson and Walker model was employed in this
150 study to represent the magnitude of the local geometric imperfections. This model is given by
151 Eq. (1).

152 $\omega_0 = 0.023 \left(\frac{\sigma_{0.2}}{\sigma_{cr}} \right) t$ (1)

153 where $\sigma_{0.2}$ is the 0.2 % proof stress of the material, σ_{cr} is the critical elastic buckling stress of
 154 the most slender plate element of the section, and t is the cross-sectional thickness.

155 Two types of analysis were performed on each FE model. First, an Eigenvalue buckling
 156 analysis was conducted to identify the critical buckling modes of the structure. These critical
 157 modes were then introduced to the non-linear FE models to perturb the mesh to account for the
 158 initial geometric imperfection patterns. Then, a geometrically and materially non-linear
 159 analysis was performed on the FE models using a modified Static Riks analysis to investigate
 160 the failure mechanism and the post-buckling behaviour of the sections.

161 2.2 Model validation

162 The shear tests conducted by Keerthan and Mahendran [4] on cold-formed steel hollow flange
 163 channel sections (LiteSteel beams) were used for the validation. The compared hollow flange
 164 sections have a shear span to clear web depth ratio (aspect ratio) of 1.0 to govern shear failure
 165 in the sections. More details of the experiments can be found in [4].

166 The experimental and FE ultimate loads ($V_{Exp.}$ and V_{FE}) are compared in Table 1. From the
 167 comparisons, it can be seen that experimental shear resistance to FE shear resistance ratio has
 168 a mean of 0.99 and a coefficient of variation (COV) of 0.039. Therefore, it is clear that the
 169 numerical models are able to predict the ultimate shear capacities of the hollow flange sections
 170 accurately.

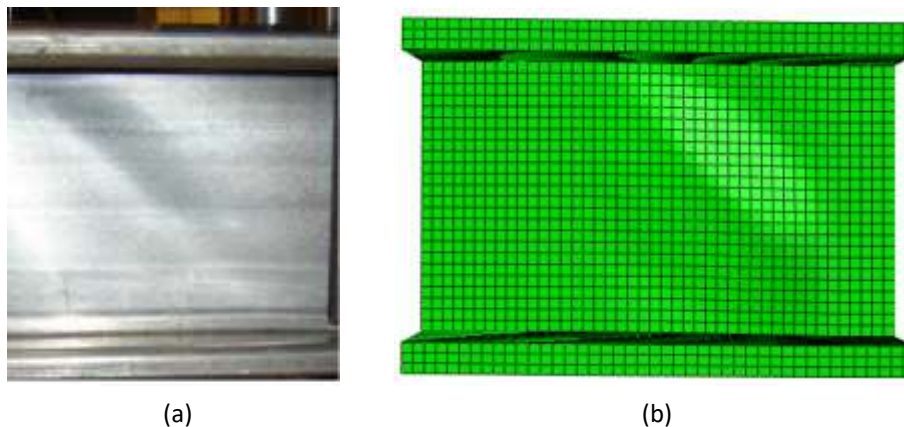
171 Table 1 Experimental [4] and FE shear resistances of LSBs

LSB section	$V_{Exp.}$ (kN)	V_{FE} (kN)	$V_{Exp.}/V_{FE}$
LSB 150×45×15×2.0	68.5	69.84	0.98
LSB 200×60×20×2.0	88.2	87.54	1.01
LSB 200×60×20×2.5	119.3	115.64	1.03
LSB 250×75×25×2.5	139.6	137.88	1.01
LSB 300×75×25×2.5	143.7	155.28	0.93
Mean			0.99
COV			0.039

172

173 The cross-section designation: *Section name Section depth (D) × Section breadth (B) × Flange*
174 *height (L) × Thickness (t)* was used to denote the considered cross-sections in this study. For
175 an instance, a rectangular hollow flange channel section (LiteSteel beam) with a depth of 150
176 mm, a breadth of 45 mm, a flange height of 15 mm and a thickness of 2.0 mm is denoted as
177 *LSB 150×45×15×2.0*.

178 In addition, the failure mechanisms were compared to further assess the FE models with the
179 experimental results. Fig. 4 illustrates the experimental and FE shear failure modes of *LSB*
180 *150×45×15×2.0* section and the comparison is found to be fairly similar. Therefore, it can be
181 concluded that the FE models simulate the shear failure mechanism of hollow flange sections
182 reasonably well.



183

184 Fig. 4 (a) Experimental [4] and (b) FE shear failure mechanisms of *LSB 150×45×15×2.0* section

185 3 Numerical parametric study

186 3.1 General

187 The influence of different cross-sectional dimensions and steel grades on the shear response of
188 cold-formed stainless steel hollow flange sections were investigated utilising the validated
189 numerical FE models. The shear response of RHFBS and THFBs were studied in this study.
190 Two section heights (150 mm, 200 mm) and three section thicknesses (1 mm, 1.5 mm, 2 mm)
191 were taken into account and four stainless steel grades including austenitic grades (1.4301,
192 1.4311) and duplex grades (1.4362, 1.4462) were considered in the study. In addition, more
193 slender 250 mm and 300 mm deep RHFBS, and 250 mm deep THFBs, of 1 mm thick and of
194 stainless steel grade 1.4462 were developed to have a wide range of FE data. Altogether, 51
195 FE models of stainless steel hollow flange beams were developed. The material properties for

196 stainless steel grades were found from EN1993-1-4 [26]. Young's modulus and Poisson's ratio
197 were taken as 200,000 MPa and 0.3, respectively. Sections with an aspect ratio of 1.0 were
198 used to govern the shear response.

199 3.2 FE shear resistances of hollow flange sections

200 The FE shear capacities of RHFBS and THFBs are summarised in Tables 2 and 3. Further, the
201 comparisons of FE shear resistances with EN19931-4 [26] and the DSM predictions, and the
202 proposed predictions are also included in tables. The details of these codified shear design
203 provisions and the details of new proposals are discussed in Section 4.

204 Table 2 Parametric study results with EN1993-1-4 [26] and the DSM predictions for RHFB sections

Section	Stainless steel grade – 1.4301					Stainless steel grade – 1.4311					Stainless steel grade – 1.4362					Stainless steel grade – 1.4462				
	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ $V_{EC3,}$ Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ $V_{DSM,}$ Proposed	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ $V_{EC3,}$ Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ $V_{DSM,}$ Proposed	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ $V_{EC3,}$ Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ $V_{DSM,}$ Proposed	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ $V_{EC3,}$ Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ $V_{DSM,}$ Proposed
RHFB 150×45×15×1.0	18.84	1.50	1.09	1.35	1.16	21.57	1.45	1.03	1.33	1.11	29.72	1.47	1.00	1.37	1.09	32.27	1.48	1.01	1.39	1.09
RHFB 150×45×15×1.5	30.15	1.31	1.02	1.21	1.02	36.29	1.32	1.01	1.16	1.03	52.22	1.36	1.01	1.22	1.05	57.18	1.38	1.01	1.24	1.06
RHFB 150×45×15×2.0	45.31	1.18	1.02	1.37	1.01	54.12	1.24	1.02	1.30	1.01	77.32	1.32	1.02	1.19	1.02	84.49	1.32	1.02	1.17	1.03
RHFB 200×60×20×1.0	21.71	1.53	1.05	1.42	1.14	27.13	1.63	1.10	1.53	1.19	33.54	1.50	0.98	1.42	1.05	36.71	1.53	1.00	1.45	1.06
RHFB 200×60×20×1.5	34.18	1.28	0.95	1.14	0.99	41.82	1.32	0.96	1.19	1.02	60.18	1.38	0.96	1.28	1.04	65.75	1.40	0.97	1.30	1.05
RHFB 200×60×20×2.0	53.08	1.30	1.01	1.20	1.01	63.82	1.30	1.00	1.15	1.01	91.39	1.34	0.99	1.20	1.03	99.82	1.35	0.99	1.22	1.04
RHFB 250×75×25×1.0																40.84	1.60	1.02	1.52	1.05
RHFB 300×120×20×1.0																45.05	1.66	1.02	1.57	1.00

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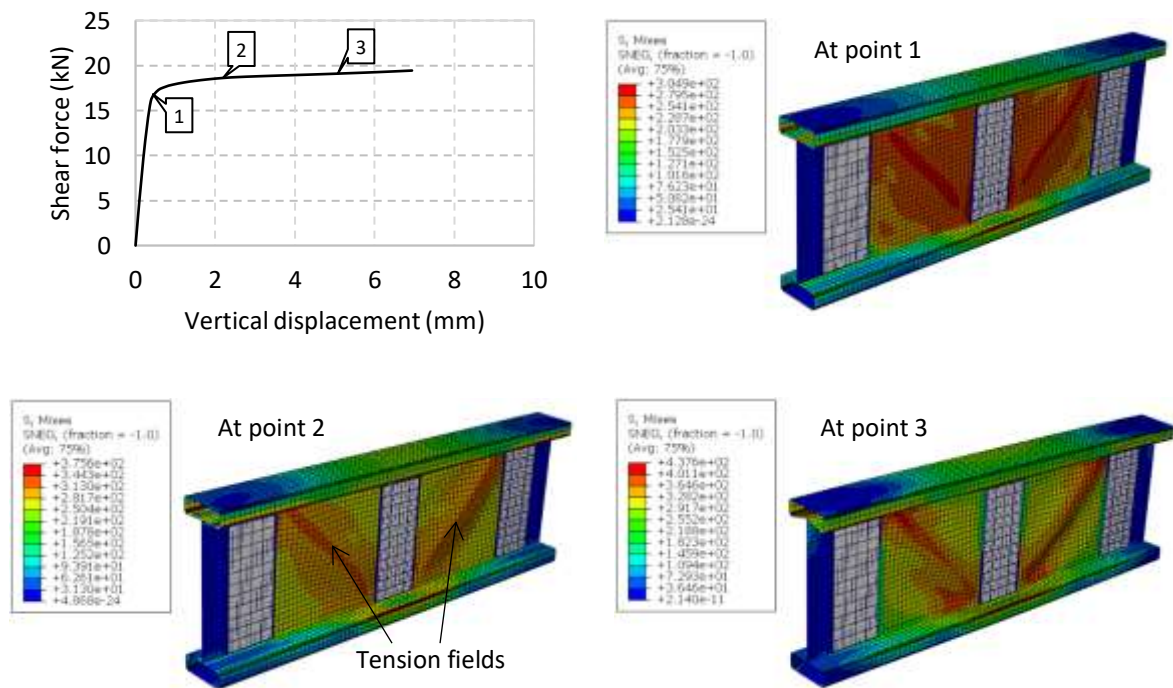
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210 Table 3 Parametric study results with EN1993-1-4 [26] and the DSM predictions for THFB sections

Section	Stainless steel grade – 1.4301					Stainless steel grade – 1.4311					Stainless steel grade – 1.4362					Stainless steel grade – 1.4462				
	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ V_{EC3} , Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ V_{DSM} , Proposed	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ V_{EC3} , Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ V_{DSM} , Proposed	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ V_{EC3} , Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ V_{DSM} , Proposed	V_{FE} (kN)	$V_{FE}/$ V_{EC3}	$V_{FE}/$ V_{EC3} , Proposed	$V_{FE}/$ V_{DSM}	$V_{FE}/$ V_{DSM} , Proposed
THFB 150×45×15×1.0	20.32	1.61	1.00	1.39	1.00	24.98	1.68	1.01	1.47	1.01	36.74	1.81	1.02	1.61	1.02	39.95	1.83	1.01	1.64	1.01
THFB 150×45×15×1.5	34.30	1.49	1.00	1.38	1.00	41.44	1.50	0.99	1.32	0.99	59.85	1.56	0.99	1.34	0.98	65.74	1.59	0.99	1.37	0.99
THFB 150×45×15×2.0	50.11	1.31	1.03	1.51	1.01	59.77	1.37	0.99	1.43	0.99	85.21	1.45	0.97	1.31	0.97	93.54	1.47	0.97	1.30	0.97
THFB 200×60×20×1.0	25.02	1.76	1.01	1.56	1.01	30.99	1.86	1.02	1.66	1.02	45.19	2.02	1.02	1.82	1.02	49.19	2.05	1.02	1.85	1.02
THFB 200×60×20×1.5	42.15	1.57	1.00	1.34	1.00	51.56	1.62	1.01	1.40	1.01	74.88	1.72	1.01	1.52	1.00	82.10	1.75	1.01	1.55	1.01
THFB 200×60×20×2.0	61.11	1.49	1.01	1.38	1.00	73.68	1.50	0.99	1.32	0.99	106.36	1.56	0.99	1.34	0.98	117.09	1.59	0.99	1.37	0.99
THFB 250×75×25×1.0																56.66	2.22	1.00	2.01	1.00

212 3.3 Results discussion

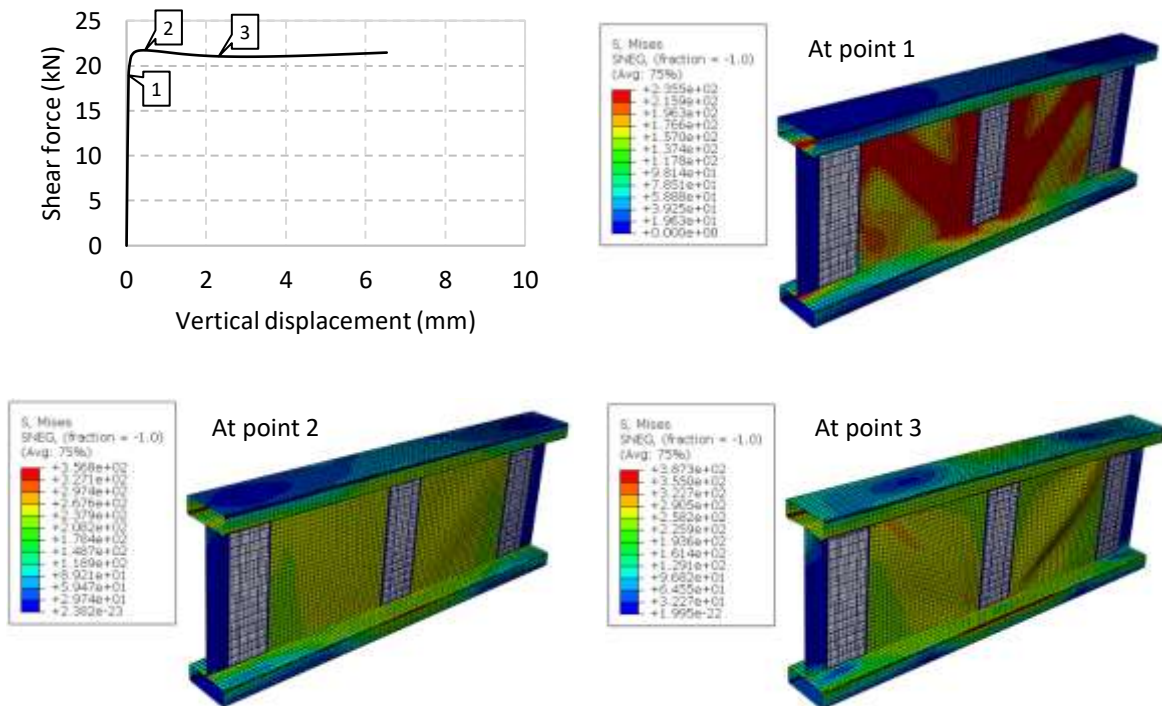
213 The shear response of cold-formed stainless steel RHFB sections and THFB sections are
 214 discussed in this section using the generated numerical FE results in the parametric study. Fig.
 215 5 illustrates the shear response of *RHFB 150×45×15×1.0* section of stainless steel grade
 216 1.4301 with its load-deflection curve while Fig. 6 shows that of *RHFB 200×60×20×1.0* section
 217 of the same steel grade. From Figs. 5 and 6, it can be seen that the out-of-plane buckling of
 218 section webs approximately start at point 1 where a change in section stiffness can be observed
 219 from the load-deflection curves. Then, the progressive buckling of both section webs at the
 220 failure point and at the post-failure regime can be observed under the shear loading. Further,
 221 the formed diagonal tension bands of highly stressed regions are clearly visible in *RHFB*
 222 *150×45×15×1.0* section as a result of the anchoring provided to the webs by the transverse
 223 web stiffeners and flanges. However, these tension fields are normalised over the section webs
 224 in *RHFB 200×60×20×1.0* section.



225

226 Fig. 5 Shear response of *RHFB 150×45×15×1.0* section at the different stages of load-deflection curve

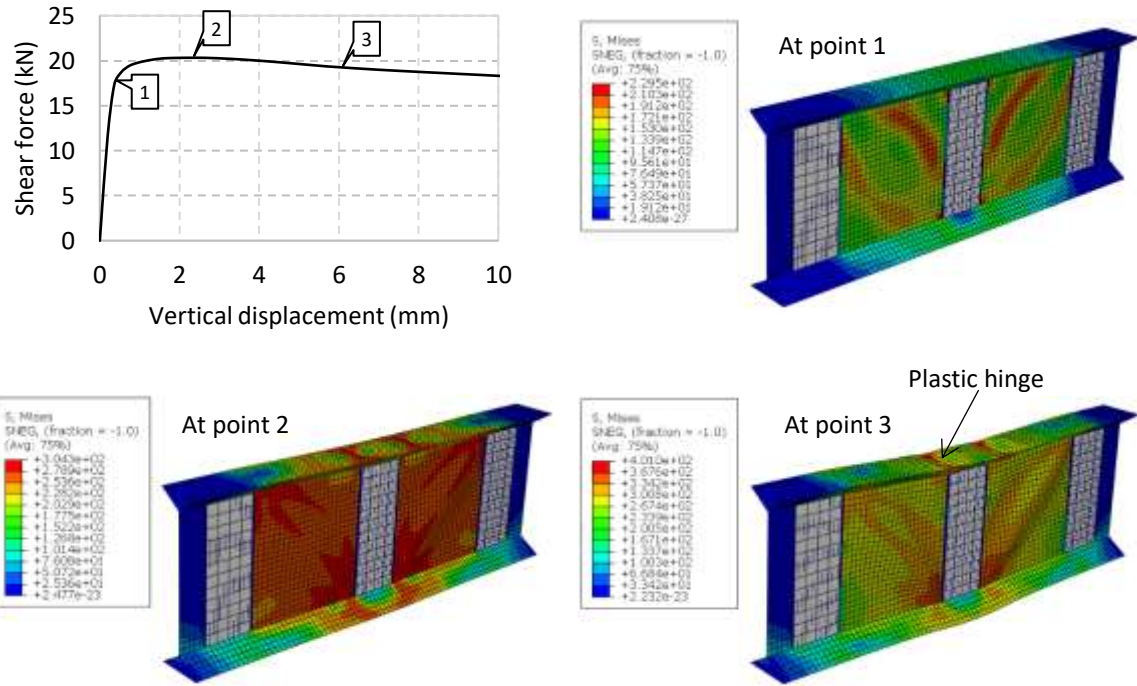
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229 Fig. 6 Shear response of *RHFB 200×60×20×1.0* section at the different stages of load-deflection curve

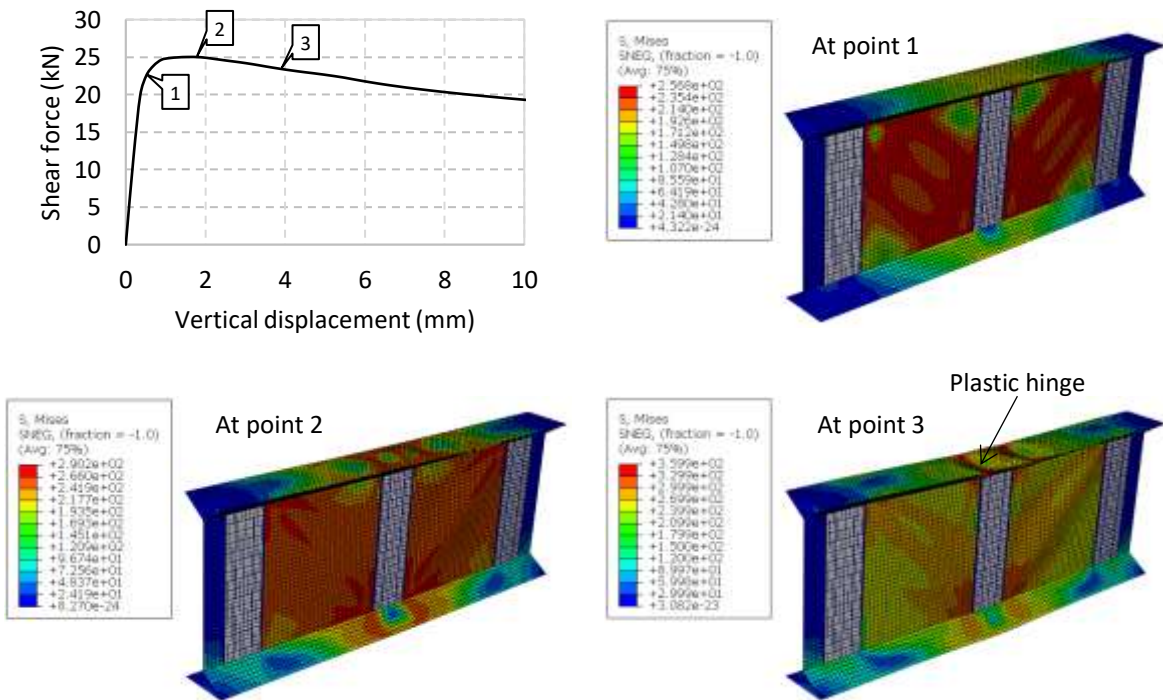
230 Figs. 7 and 8 illustrate the shear behaviour of *THFB 150×45×15×1.0* section and *THFB*
 231 *200×60×20×1.0* section of stainless steel grade 1.4301, respectively with their load-deflection
 232 curves. Both sections begin to show signs of out-of-plane buckling of webs at around point 1
 233 of their load-deflection curves. After this, the progression of web shear buckling of both
 234 sections can be observed through their failure points. The increased anchoring facilitated by
 235 the triangular flanges and transverse web stiffeners caused the distribution of the stresses in the
 236 webs more evenly. Therefore, the diagonal tension bands are not clearly visible in THFB
 237 sections as opposed to RHFB sections. Moreover, a plastic hinge type mechanism is formed in
 238 the mid-span of THFB sections at the post-failure region. The excessive compression stresses
 239 induced within the triangular top flanges as a result of the anchoring provided by the top flanges
 240 to the tension fields could lead to this formation.



241

242 Fig. 7 Shear response of THFB 150×45×15×1.0 section at the different stages of load-deflection curve

243



244

245 Fig. 8 Shear response of THFB 200×60×20×1.0 section at the different stages of load-deflection curve

246 4 Assessment of shear design rules

247 The generated numerical database of hollow flange sections was incorporated in this section to
248 evaluate the shear design rules provided in European standards for stainless steel [26] and the
249 DSM shear design rules. Following the assessment of codified shear provisions, new shear
250 design equations were proposed using FE results.

251 4.1 European standards for stainless steel, EN1993-1-4 [26]

252 European standards for stainless steel [26] adopts the shear design rules provided in European
253 standards for plated steel, EN1993-1-5 [38]. According to that, the summation of the shear
254 buckling resistance of the section web ($V_{bw,Rd}$) and the flange contribution to the shear
255 resistance of the section ($V_{bf,Rd}$) gives the shear resistance of the section ($V_{b,Rd}$) as expressed in
256 Eq. (2).

$$257 V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (2)$$

258 where the parameter η takes into account the strain hardening of stainless steel, γ_{M1} is the partial
259 safety factor, f_{yw} is the yield strength of the web, h_w is the depth of the web, and t_w is the
260 thickness of the web.

261 The shear buckling resistance of the web ($V_{bw,Rd}$) is given by Eq. (3) in which χ_w is the web
262 shear buckling reduction factor.

$$263 V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (3)$$

264 The flange contribution ($V_{bf,Rd}$) is defined by Eq. (4).

$$265 V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) \quad (4)$$

266 where b_f is the width of the flange, t_f is the thickness of the flange, and f_{yf} is the yield strength
267 of the flange. M_{Ed} is the design bending moment of the section and $M_{f,Rd}$ is the moment
268 resistance of the flanges alone. The parameter c is the distance to the location of the plastic
269 hinge from the transverse stiffener. Eq. (5) is given in EN1993-1-4 [26] to calculate the
270 parameter c .

$$271 c = a \left[0.17 + \frac{3.5 b_f t_f^2 f_{yf}}{t_w h_w^2 f_{yw}} \right] \text{ and } \frac{c}{a} \leq 0.65 \quad (5)$$

272 where a is the length of the shear panel.

273 Two sets of expressions are set out in EN1993-1-4 [26] to calculate the web shear buckling
274 reduction factor (χ_w) of the section webs with and without rigid end posts. These expressions
275 for the webs with rigid end posts are given by Eqs. (6)-(8).

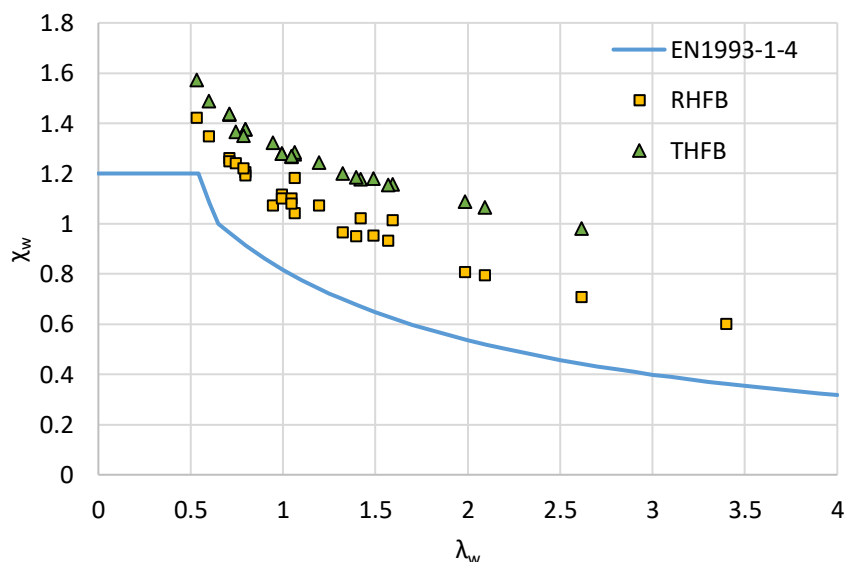
276
$$\chi_w = \eta \text{ for } \bar{\lambda}_w \leq 0.65/\eta \tag{6}$$

277
$$\chi_w = 0.65/\bar{\lambda}_w \text{ for } 0.65/\eta < \bar{\lambda}_w < 0.65 \tag{7}$$

278
$$\chi_w = 1.56/(0.91 + \bar{\lambda}_w) \text{ for } \bar{\lambda}_w \geq 0.65 \tag{8}$$

279 where $\bar{\lambda}_w$ is the slenderness of the web.

280 The EN1993-1-4 [26] shear design rules were then evaluated using the numerical FE results
281 generated in Section 3 to assess their applicability to predict the shear resistance of cold-formed
282 stainless steel hollow flange sections. The comparison of EN1993-1-4 [26] shear design rules
283 with FE results for each section is given in Tables 2 and 3. The generated numerical shear
284 capacities are plotted with EN1993-1-4 [26] web shear buckling reduction factor (χ_w) in Fig. 9
285 and can be seen that the codified shear provisions are too conservative for cold-formed stainless
286 steel hollow flange sections. Further, THFBs are found to have higher shear resistances than
287 RHFBs.



288

289 Fig. 9 Comparison of FE shear capacities with the web shear buckling reduction factor (χ_w) of EN1993-1-4 [26]

290 Table 4 summarises the overall mean and COV of FE shear resistance to predicted shear
291 resistance ratio for each cross-section type. The conservative nature of EN1993-1-4 [26] shear

292 capacity predictions for hollow flange sections is further confirmed from mean and COV
 293 values. Eurocode provisions do not take into account the favourable effect of fixity at the web-
 294 to-flange juncture of the hollow flange sections to shear buckling resistance of the section web
 295 could be one reason for these conservative predictions.

296 Table 4 Overall mean and COV of FE to predicted shear resistance ratio for each section type

	EN1993-1-4 [26]		DSM	
	Current	Proposed	Current	Proposed
RHFBs				
Mean	1.40	1.01	1.30	1.05
COV	0.087	0.034	0.097	0.048
THFBs				
Mean	1.66	1.00	1.49	1.00
COV	0.132	0.015	0.127	0.015

297

298 Therefore, Eurocode shear provisions were modified to enhance the shear resistance prediction
 299 accuracy of stainless steel hollow flange sections. The new set of expressions for web shear
 300 buckling reduction factor (χ_w) of EN1993-1-4 [26] were proposed using numerical FE shear
 301 capacities of hollow flange sections and following regression analyses. The elastic shear
 302 buckling coefficients proposed for RHFBs and THFBs by Keerthan and Mahendran [22] were
 303 utilised here when modifying the codified expressions. Therefore, proposed shear provisions
 304 do take into account the available fixity at the web-to-flange juncture.

305 The proposed expressions for web shear buckling reduction factor (χ_w) of RHFBs are given by
 306 Eqs. (9)-(11).

$$307 \chi_w = 1.4 \text{ for } \bar{\lambda}_w \leq 0.5 \quad (9)$$

$$308 \chi_w = 1.08/\bar{\lambda}_w^{0.34} \text{ for } 0.5 < \bar{\lambda}_w < 1.25 \quad (10)$$

$$309 \chi_w = 2.75/(1.5 + \bar{\lambda}_w) \text{ for } \bar{\lambda}_w \geq 1.25 \quad (11)$$

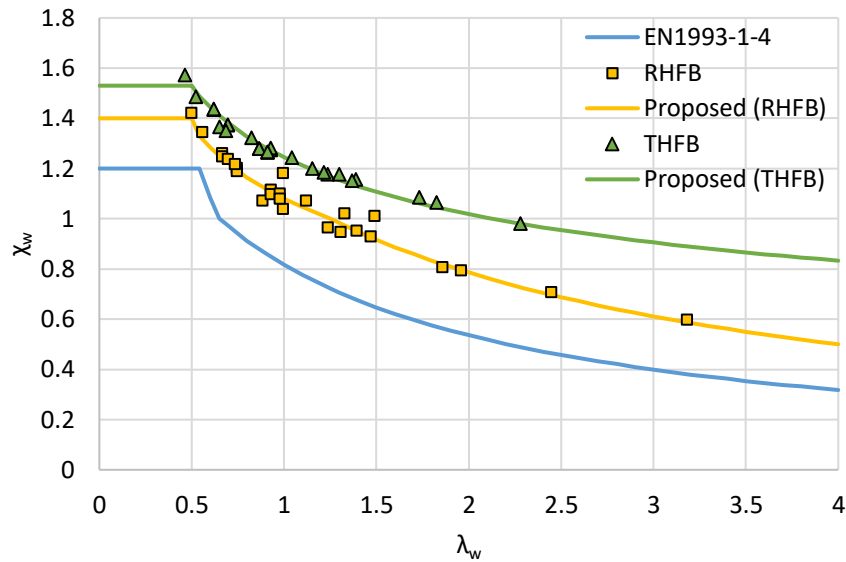
310 Eqs. (12) and (13) provides the modified expressions for web shear buckling reduction factor
 311 (χ_w) of THFBs.

$$312 \chi_w = 1.53 \text{ for } \bar{\lambda}_w \leq 0.5 \quad (12)$$

$$313 \chi_w = 1.245/\bar{\lambda}_w^{0.29} \text{ for } 0.5 < \bar{\lambda}_w \quad (13)$$

314 Fig. 10 plots the proposed expressions for web shear buckling reduction factor (χ_w) for stainless
 315 steel hollow flange sections with FE shear capacities. It can be seen that the proposed curves

316 are fitted well with the distribution of the corresponding FE results, therefore, suggesting better
 317 prediction accuracy over the codified web shear buckling curve of EN1993-1-4 [26]. The mean
 318 and COV of proposed EN1993-1-4 [26] provisions given in Table 4 also implies the improved
 319 shear resistance predictions for both section types over the current shear provisions.



320
 321 Fig. 10 Comparison of FE shear capacities with the proposed web shear buckling reduction factor (χ_w) for
 322 EN1993-1-4 [26]

323 4.2 The direct strength method

324 The DSM has been developed as an alternative design approach to the traditional cross-section
 325 classification framework known as the effective width method. The clause 7.2.3.3 of Australian
 326 and New Zealand standards, AS/NZS 4600 [24] includes the details of the DSM shear design
 327 rules for the sections with transverse web stiffeners.

328 The sectional shear capacity (V_v) according to the DSM is given by Eqs. (14) and (15).

$$329 V_v = V_y \text{ for } \lambda \leq 0.776 \quad (14)$$

$$330 V_v = \left[1 - 0.15 \left(\frac{1}{\lambda^2} \right)^{0.4} \right] \left(\frac{1}{\lambda^2} \right)^{0.4} V_y \text{ for } \lambda > 0.776 \quad (15)$$

331 where λ is the cross-sectional slenderness.

332 The slenderness of the cross-section, λ is defined as in Eq. (16) using the shear yield capacity
 333 of the section (V_y) and the elastic shear buckling capacity of the section (V_{cr}).

334
$$\lambda = \sqrt{\frac{V_y}{V_{cr}}} \tag{16}$$

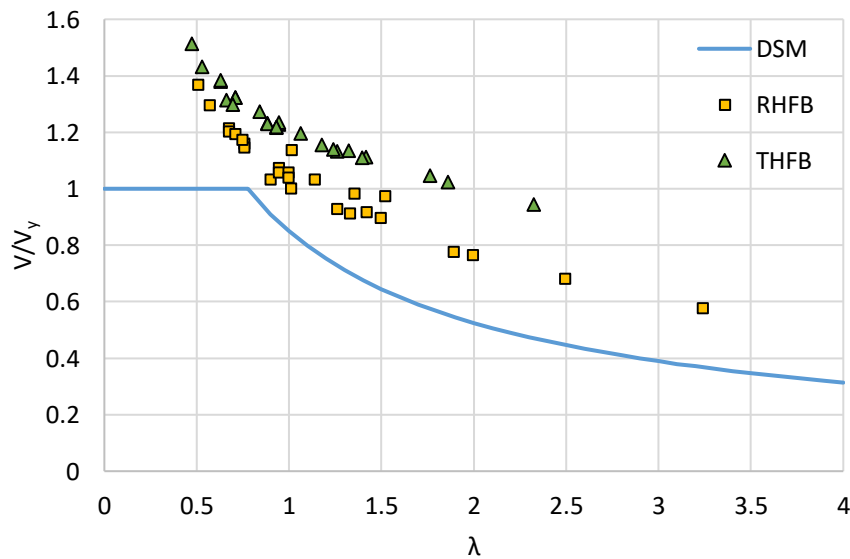
335 Eqs. (17) and (18) can be used to calculate the shear yield capacity (V_y) and the elastic shear
 336 buckling capacity (V_{cr}) of the section.

337
$$V_y = 0.6 f_{yw} d_1 t_w \tag{17}$$

338
$$V_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)} \frac{t_w^3}{d_1} \tag{18}$$

339 where f_{yw} is the yield strength of the web, d_1 is the flat depth of the web, t_w is the thickness of
 340 the web, E is Young's modulus, ν is Poisson's ratio, and k is the elastic shear buckling
 341 coefficient of the section.

342 The applicability of the DSM shear design provisions to predict the section capacities of cold-
 343 formed stainless steel hollow flange sections were then assessed using the numerical parametric
 344 study results gathered in Section 3. The elastic shear buckling coefficient (k) of the hollow
 345 flange sections were found from Keerthan and Mahendran [22]. Fig. 11 illustrates the FE shear
 346 capacities of RHFBs and THFBs together with the DSM shear design curve. Moreover, the
 347 overall mean and COV of FE shear capacity to DSM predicted shear capacity ratio for each
 348 hollow flange section type is given in Table 4. Both these comparisons reflect that the DSM
 349 shear design provisions significantly under-predict the section capacities of stainless steel
 350 RHFBs and THFBs as similar to EN1993-1-4 [26] shear design provisions.



351

352 Fig. 11 Comparison of FE shear capacities with the DSM shear design curve

353 Following the assessment of DSM shear design rules, modifications were made to Eqs. (16)
354 and (17) aiming to achieve improved shear capacity predictions for the cold-formed stainless
355 steel hollow flange sections. Regression analyses were conducted to fit the proposed DSM
356 curves to FE shear capacities.

357 The proposed DSM equations for stainless steel RHFB sections are expressed in Eqs. (19)-
358 (21).

$$359 \quad V_v = 1.36V_y \text{ for } \lambda \leq 0.5 \quad (19)$$

$$360 \quad V_v = \frac{V_y}{\lambda^{0.444}} \text{ for } 0.5 < \lambda \leq 1.0 \quad (20)$$

$$361 \quad V_v = \left[1 - 0.01 \left(\frac{1}{\lambda^2} \right)^{0.232} \right] \left(\frac{1}{\lambda^2} \right)^{0.232} V_y \text{ for } \lambda > 1.0 \quad (21)$$

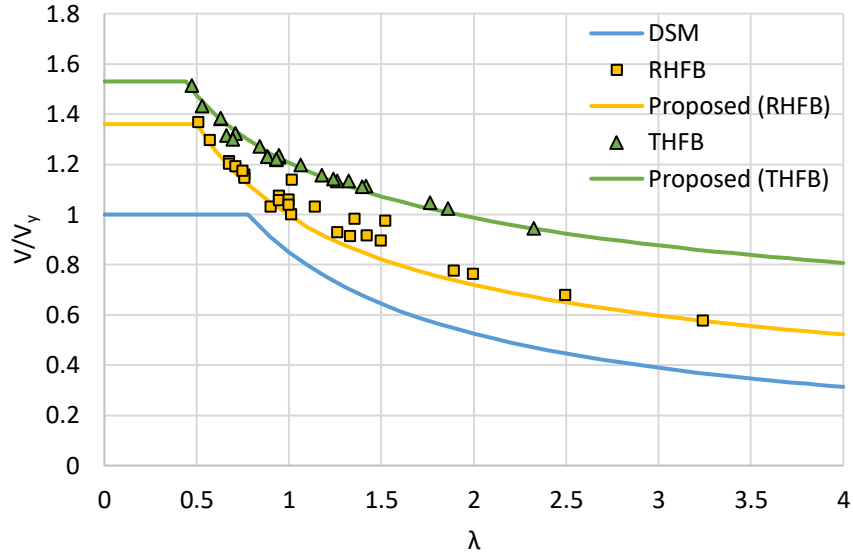
362 Eqs. (22) and (23) provide the proposed DSM equations for stainless steel THFB sections.

$$363 \quad V_v = 1.53V_y \text{ for } \lambda \leq 0.44 \quad (22)$$

$$364 \quad V_v = \frac{1.206V_y}{\lambda^{0.29}} \text{ for } \lambda > 0.44 \quad (23)$$

365 The new DSM equations and existing DSM equations for shear are plotted together with the
366 FE capacities of stainless steel RHFBs and THFBs in Fig. 12. The comparison shows that the
367 proposed DSM curves follow the distribution of the respective FE results well. Further, the
368 mean and COV of proposed DSM provisions given in Table 4 suggest enhanced capacity
369 predictions for stainless steel hollow flange sections over the current DSM shear design rules.

370



371

372 Fig. 12 Comparison of FE shear capacities with the proposed DSM shear design curve

373 4.3 Reliability analysis

374 Reliability analysis was conducted for the proposed EN1993-1-4 [26] and the DSM resistance
 375 models according to North American specifications for cold-formed steel [23]. The capacity
 376 reduction factor (ϕ_v) of each resistance model was calculated using Eq. (24).

377
$$\phi_v = 1.52M_m F_m P_m e^{-\beta_0 \sqrt{(V_m^2 + V_f^2 + C_p V_p^2 + V_q^2)}} \quad (24)$$

378 where $M_m=1.1$ and $V_m=0.1$ are mean and COV of the material factor, respectively. $F_m=1.0$ and
 379 $V_f=0.05$ are mean and COV of the fabrication factor, respectively. P_m and V_p (not less than
 380 0.065) are mean and COV of the actual (FE) resistance to predicted resistance ratio,
 381 respectively. β_0 is the target reliability index and $V_q=0.21$ is the COV of the load effect.

382 The correction factor, C_p is given by Eq. (25).

383
$$C_p = \left[1 + \frac{1}{n} \right] \left[\frac{m}{m-2} \right] \quad (25)$$

384 where $m=n-1$ and n is the total number of data.

385 For the calculations, the target reliability index, β_0 was taken as 2.5 and the minimum
 386 recommended value was assigned for V_p as the actual values were found to be less than 0.065.
 387 The calculated capacity reduction factors for the proposed EN1993-1-4 [26] resistance models
 388 are 0.91 for RHFb's and 0.90 for THFB's. For the proposed DSM resistance models, the
 389 calculated capacity reduction factors are found to be 0.95 for RHFb's and 0.90 for THFB's.

390 Therefore, a value of 0.90 is recommended in general for the capacity reduction factor of all
391 the resistance functions.

392 **5 Concluding remarks**

393 The shear response of cold-formed stainless steel hollow flange sections was investigated using
394 numerical analysis in this paper. The numerical parametric studies were conducted for RHFB
395 sections and THFB sections using the validated FE models. Various influential parameters such
396 as the height of the section, the thickness of the section and the steel grade were taken into
397 account in the study and 51 FE models of hollow flange sections were developed. The
398 numerical results were used to observe the shear response of the sections and to evaluate the
399 codified shear provisions. From the FE results, it can be observed that diagonal tension fields
400 are formed within section webs of RHFB sections however more even distribution of the
401 stresses can be seen in the webs of THFB sections with no clearly visible tension bands as a
402 result of increased anchoring provided by the flanges. The increased anchoring provided by the
403 flanges results into developing plastic hinge type mechanism in the top flanges of THFB
404 sections at the mid-span. Moreover, the shear resistance of THFBs is found to be relatively
405 higher than RHFBs. In general, the evaluation of EN1993-1-4 [26] and the DSM shear design
406 rules using the generated numerical results suggests that the current codified provisions
407 considerably under-predict the shear resistance of stainless steel hollow flange sections.
408 Therefore, modifications were proposed to the codified provisions aiming improved shear
409 capacity predictions. The proposed shear provisions offer more accurate and consistent shear
410 capacity predictions over the codified provisions. The reliability of the proposed provisions
411 was also assessed.

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