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# **Numerical Modelling and Shear Design Rules of Stainless Steel Lipped Channel Sections**

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## **Abstract**

The demand for highly structurally efficient stainless steel is limited to a certain extent by its high initial cost. Therefore, the utilisation of material to the optimum possible level is important. In achieving this, further consideration should be given to enhance the design rules where beneficial effects such as pronounced strain hardening in stainless steel should be taken into account in the design process. In addition to that, a thorough understanding of the structural behaviour of stainless steel sections is also required. However, the shear behaviour and capacity of cold-formed stainless steel lipped channel beams (LCBs) have not been thoroughly investigated previously. Therefore, experimental and detailed finite element (FE) modelling were undertaken to investigate the shear behaviour and strength of stainless steel LCBs. A comprehensive parametric study was also conducted by developing 100 FE models. From the results, the available post-buckling strength in slender stainless steel LCBs was highlighted.

33 Furthermore, the beneficial strength increment due to the strain hardening effect of stainless  
34 steel, particularly for compact LCBs in shear, was investigated. Comparisons indicated that  
35 current EN1993-1-4 and direct strength method (DSM) shear design rules are too conservative  
36 in particularly for compact sections. Thus, existing shear design rules were modified to enhance  
37 the overall prediction accuracy for stainless steel LCBs while attention was given to capture  
38 the available inelastic reserve capacity.

39 *Keywords: Cold-formed stainless steel; Lipped channel beams; Finite element modelling;*  
40 *Shear design rules; EN1993-1-4; Direct strength method*

## 41 **1 Introduction**

42 Stainless steel is becoming a highly demanding construction material (see Figure 1 for  
43 application of stainless steel in structures [1]). This is primarily due to its improved  
44 characteristics as a result of the well-controlled alloying composition of each stainless steel  
45 grade. Thus stainless steel usually exhibits appealing characteristics such as higher strength-  
46 to-weight ratio, high ductility, impact resistance, fire resistance and good corrosion resistance  
47 thus featuring greater durability and low maintenance cost, and also recyclability in addition to  
48 its aesthetically pleasing good finish. The chromium content of stainless steel is more than 10.5  
49 % and it contributes to form a chromium-rich oxide layer on the surface of stainless steel [2].  
50 This is the main reason for its high corrosion resistant. However, these benefits have come to  
51 a cost due to the alloying composition (chromium and nickel) of stainless steel, thus, the  
52 material usage should be optimised by giving more attention to the design process of stainless  
53 steel structural members.



54

55

Figure 1: Gent Sint Pieters railway station, Belgium [1].

56 Over the past few years, research into stainless steel sections has covered numerous cross-  
57 section types. Available stub column tests of square, rectangular and circular hollow sections,  
58 I-sections, angle sections, and channel sections have been gathered and reported alongside with  
59 bending tests of the above mentioned hollow sections and I-sections by Gardner and  
60 Theofanous [3]. Channels under minor axis bending have been studied by Theofanous et al. [4]  
61 while channels under combined loading have been investigated by Liang et al. [5,6].  
62 Furthermore, angles, channels and T-sections in bending about an axis that is not symmetry  
63 have been covered by Zhao and Gardner [7]. Moreover, distortional-global interaction buckling  
64 of stainless steel lipped channel sections have been investigated in [8,9] while a recent research  
65 provides the details of major axis bending behaviour of lipped channel sections [10]. In  
66 addition, previous studies have been conducted on the shear behaviour of cold-formed steel  
67 channel sections by Keerthan and Mahendran [11–13]. Furthermore, studies have been  
68 conducted on the combined bending and shear behaviour of high strength cold-formed steel C-  
69 sections and purlins by Pham and Hancock [14,15]. However, it is worth to note that there is  
70 no comprehensive study available for shear behaviour of stainless steel lipped channel sections.

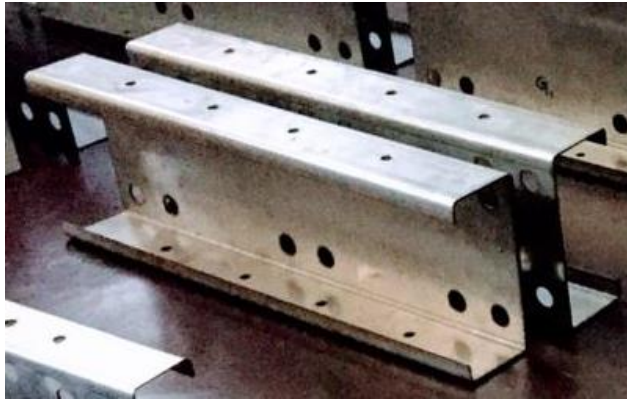
71 Currently, available design guidelines for stainless steel sections include European codes such  
72 as EN1993-1-4 [16] and EN1993-1-5 [17], Australian/New Zealand standard AS/NZS 4673  
73 [18], and American specification SEI/ASCE-8 [19]. These design guidelines are in accordance  
74 with the conventional carbon steel design guidelines, thus utilise the elastic, perfectly-plastic  
75 material models [20] limiting the ultimate strength to yield stress of the material, which is not  
76 true for stainless steel as it shows a non-linear stress-strain behaviour due to its pronounced  
77 strain hardening effect. In addition, these design guidelines are based on the conventional cross-  
78 section classification approach, known as the effective width method, which considers cross  
79 sections as an assemblage of plate elements [21]. However, it has been proved that there is a  
80 considerable post-buckling strength in channel sections due to the element interaction  
81 presented at the web-flange juncture [13]. Therefore, the main concern in the design process  
82 should be given to the pronounced strain hardening effect of stainless steel which emphasises  
83 the continuation of strength beyond yield stress, and to the requirement of accounting for  
84 element interaction. In order to address these shortcomings in the current design guidelines,  
85 advanced design approaches, such as the continuous strength method (CSM) and the direct  
86 strength method (DSM), have been proposed.

87 In this paper, the shear behaviour of stainless steel LCBs is investigated with scope to improve  
88 the shear capacity prediction accuracy using EN1993-1-4 [16] and DSM. The application of  
89 CSM to predict the shear capacity of stainless steel LCBs is not investigated herein, thus  
90 recommended as future work. Conducted experiments were utilised to develop accurate and  
91 reliable finite element (FE) models of stainless steel LCBs, details of which are elaborated in  
92 the paper. In order to collect a comprehensive database on the shear behaviour of stainless steel  
93 LCBs, a detailed parametric study was conducted following the validation of the FE models.  
94 Common austenitic and duplex stainless steel grades and both compact and slender cross  
95 sections were considered. Improved shear design equations are presented while confirming  
96 their prediction accuracy. Moreover, pronounced inelastic reserve capacity in compact stainless  
97 steel LCBs is highlighted and attempts were made to capture this in capacity prediction  
98 equations.

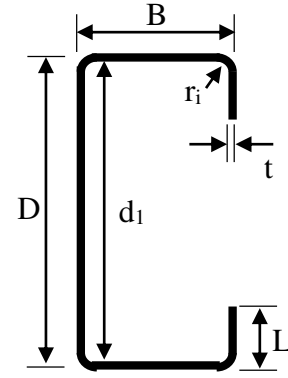
## 99 **2 Experimental study**

100 To study the shear behaviour of stainless steel LCBs a testing programme was conducted. The  
101 testing programme was comprised of nine cold-formed stainless steel lipped channel sections  
102 made of austenitic stainless steel grade 1.4301. Three sectional geometries (with section depths  
103 of 100 mm, 150 mm and 200 mm) with three different section thicknesses (1.2 mm, 1.5 mm  
104 and 2.0 mm) were chosen to represent a range of slenderness values. Figure 2 shows the tested  
105 stainless steel lipped channel sections and the notations used for the cross-sectional dimensions  
106 where  $D$  is the section depth,  $B$  is the width of the flange,  $L$  is the depth of the lip,  $d_1$  is the  
107 clear web depth,  $t$  is the thickness and  $r_1$  is the internal corner radius. All LCB cross sections  
108 are denoted as LCB  $D \times B \times L \times t$  where LCB stands for Lipped Channel Beam followed by the  
109 nominal section dimensions in millimetres (section depth  $D \times$  flange width  $B \times$  lip depth  $L \times$   
110 section thickness  $t$ ). Table 1 provides the measured cross sectional dimensions of the stainless  
111 steel LCBs employed in the experimental programme.

112



(a)



(b)

113 Figure 2: (a) Tested stainless steel lipped channel sections; (b) Cross section details.

114

115 Table 1: Dimensions of the tested stainless steel LCBs.

<i>Section</i>	$d_1$ (mm)	$B$ (mm)	$L$ (mm)	$t$ (mm)	$r_i$ (mm)
LCB 100×50×15×1.2	97.5	50	16.5	1.18	2.0
LCB 100×50×15×1.5	97	50.25	16.25	1.5	2.0
LCB 100×50×15×2.0	95.5	50.25	16.5	1.99	2.0
LCB 150×65×15×1.2	147	65.5	16	1.18	2.0
LCB 150×65×15×1.5	147	66	16.5	1.5	2.0
LCB 150×65×15×2.0	146.5	65.5	16	1.99	2.0
LCB 200×75×15×1.2	197	75.5	16.25	1.18	2.0
LCB 200×75×15×1.5	198	76.75	15	1.5	2.0
LCB 200×75×15×2.0	197	75.5	15.5	1.99	2.0

116

117 To obtain the mechanical properties of the used stainless steel grade, tensile coupon tests were  
 118 conducted. Coupons were extracted from the middle part of the web and flanges of the sections  
 119 covering all the sections used here. Coupons were tested at a uniform strain rate of  $0.0005 \text{ s}^{-1}$ .  
 120 Obtained mechanical properties were utilised in the development of the finite element models  
 121 to validate the experiments. Average values of Young's modulus ( $E$ ), 0.01% proof stress ( $\sigma_{0.01}$ ),  
 122 0.2% proof stress ( $\sigma_{0.2}$ ), ultimate tensile stress ( $\sigma_u$ ), Ramberg-Osgood parameters  $n$  and  $m$ ,  
 123 strain corresponding to the ultimate tensile stress ( $\epsilon_u$ ) and strain at fracture ( $\epsilon_f$ ) are listed in  
 124 Table 2.

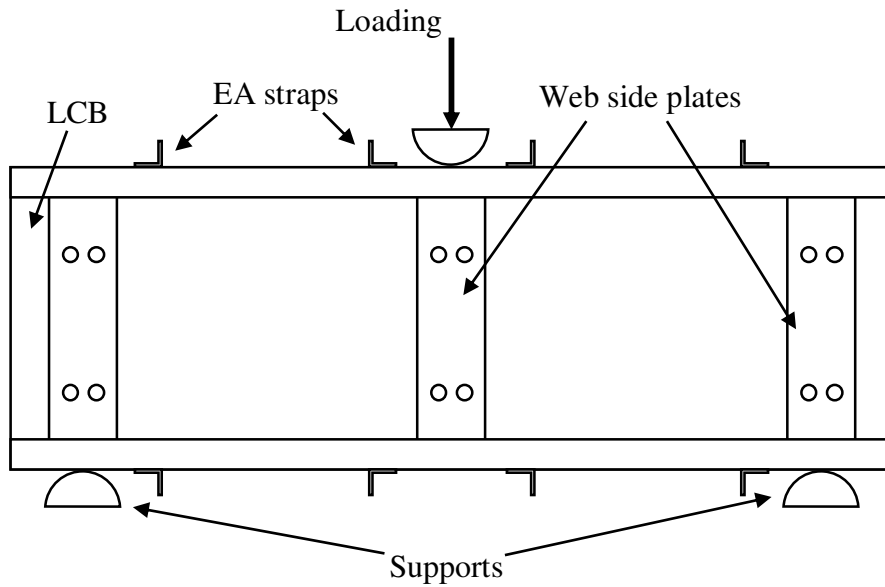
125 Table 2: Average material properties of the stainless steel grade 1.4301 extracted from tensile  
 126 coupon test.

	$E$ (GPa)	$\sigma_{0.01}$ (MPa)	$\sigma_{0.2}$ (MPa)	$\sigma_u$ (MPa)	$n$	$m$	$\epsilon_u$	$\epsilon_f$
Average value	197.3	161.2	253.9	725.3	6.6	1.98	0.54	0.61

127

128 LCB is a mono-symmetric open section thus an unbalanced shear flow presents within its cross  
 129 section. Therefore, in the experimental setup back-to-back LCBs were used in order to  
 130 eliminate any torsional effects. Two LCBs were attached together back-to-back using three T-  
 131 shaped stiffeners in between them at the two ends and at the mid-span. All the specimens were  
 132 subjected to three-point loading configuration by applying a point load at the mid-span of the  
 133 simply supported back-to-back beam setup. Figure 3 illustrates the three-point loading  
 134 arrangement while Figure 4 shows the back-to-back LCBs setup used in the testing.

135



136

137 Figure 3: Schematic diagram of three-point loading arrangement.

138

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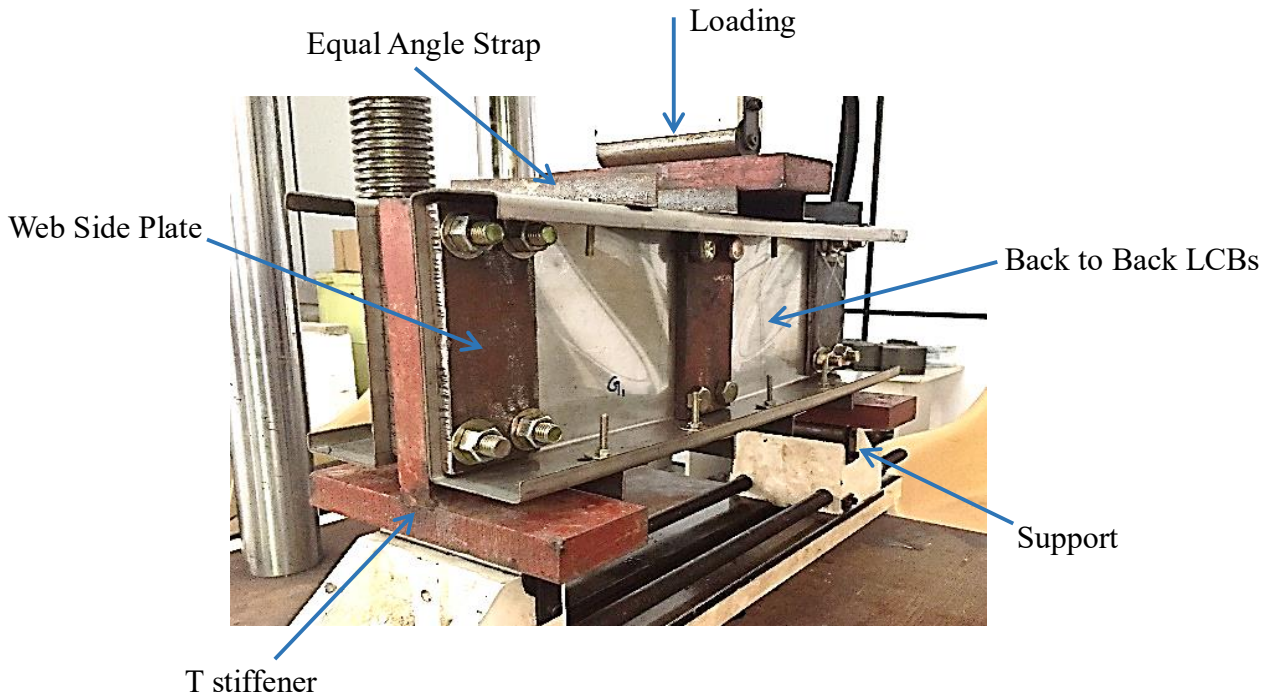


Figure 4: Experimental setup.

141

142

143 Displacement control was employed in the loading head with a constant downward moving  
 144 rate of 0.7 mm/min. At the mid-span, loading head was attached to the T-stiffener. Then the  
 145 load was transferred to two specimens. 10 mm thick full depth rigid plates were attached to the  
 146 specimen webs at the mid-span and at the two ends to avoid any web bearing failure. At the  
 147 beam ends, a pin and a roller support were assigned to the T-stiffeners to simulate simply  
 148 supported conditions. A 30 mm gap was maintained between two LCBs in the back-to-back  
 149 setup using T-stiffeners. Due to this, two LCBs were able to behave independently while  
 150 remaining as one unit to resist torsional effects. Spacing between two vertical rows of bolts at  
 151 each rigid plate was 45 mm. At the supports, a 25 mm overhang was kept to the beam edge  
 152 from the outer bolt row.

153 Equal angle straps were attached to the both top and bottom flanges adjacent to the supports  
 154 and to the loading point. The purpose of this straps were to prevent any distortional buckling  
 155 that the sections could undergo. Keerthan and Mahendran [12] showed that the shear capacity  
 156 of a section is not affected by the bending stresses for sections with shorter spans (with an  
 157 aspect ratio=1.0) while combined bending and shear interaction should be considered for  
 158 sections with relatively longer spans. Therefore, all the LCBs employed in the testing  
 159 programme had relatively shorter spans with an aspect ratio (shear span (a)/ clear web depth



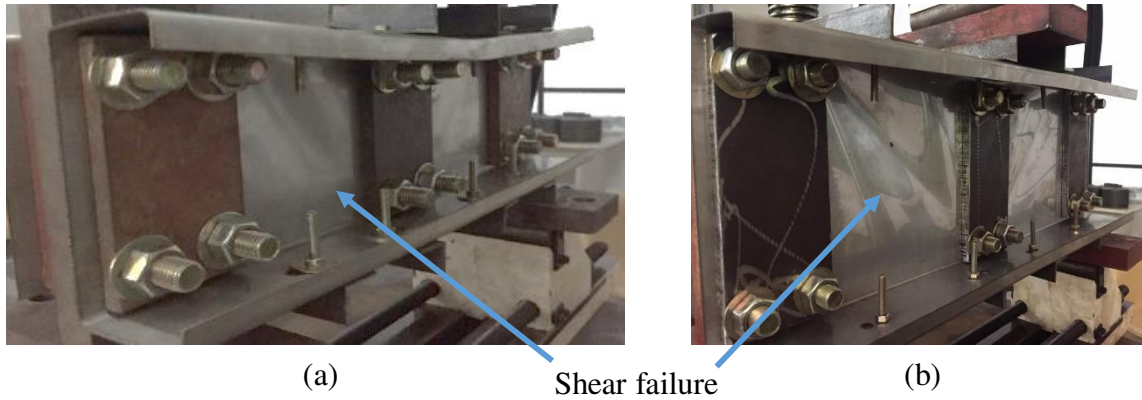
160 (d<sub>1</sub>) of 1.0 in order to govern the shear failure mode and to suppress the bending failure mode.  
 161 Vertical displacements of the LCBs were measured at the mid-span by using two Linear  
 162 Variable Differential Transducers (LVDTs).

163 Table 3: Ultimate loads and shear capacities obtained from experiments for stainless steel  
 164 sections.

<i>Section</i>	$P_T$ (kN)	$V_T = P_T / 4$ (kN)
LCB 100×50×15×1.2	74.0	18.5
LCB 100×50×15×1.5	97.8	24.4
LCB 100×50×15×2.0	144.0	36.0
LCB 150×65×15×1.2	86.4	21.6
LCB 150×65×15×1.5	105.1	26.3
LCB 150×65×15×2.0	174.2	43.6
LCB 200×75×15×1.2	91.9	23.0
LCB 200×75×15×1.5	105.9	-
LCB 200×75×15×2.0	188.2	47.1

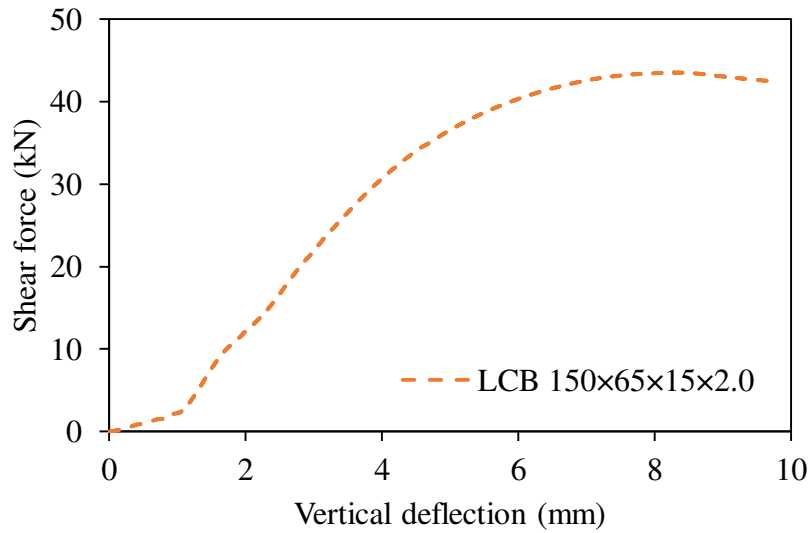
165  
 166 Table 3 summarises the ultimate peak loads ( $P_T$ ) recorded in the experiments for all the nine  
 167 LCBs with the calculated ultimate shear capacities ( $V_T$ ). For LCB 200×75×15×1.5 specimen,  
 168 premature failure was observed during the test as the bolts at the loading point were failed due  
 169 to the yielding. Therefore, this test result was not considered in the validation process and  
 170 excluded from the design calculations. All the tests conducted exhibited shear failure modes as  
 171 expected and shear failure modes of LCB 150×65×15×2 and LCB 200×75×15×1.2 specimens  
 172 are illustrated in Figure 5. Load-deflection curve for LCB 150×65×15×2.0 section is shown in  
 173 Figure 6 as recorded during the testing programme.

174



175 Figure 5: Shear failure modes (a) LCB 150×65×15×2; (b) LCB 200×75×15×1.2.

176



177 Figure 6: Load-deflection curve of LCB 150×65×15×2.0 section.

178

### 179 3 Finite Element (FE) modelling

#### 180 3.1 General

181 This section provides the details of the development of FE models which were then used to  
 182 investigate the shear behaviour of LCBs. A detailed parametric study was conducted using the  
 183 developed FE models following the validation process, details of which are presented in the  
 184 next section. For the validation of the FE models, experimental results of both stainless steel  
 185 and cold-formed steel sections were employed. The details of cold-formed steel sections  
 186 employed in the validation process can be found from Keerthan and Mahendran [12]. For the

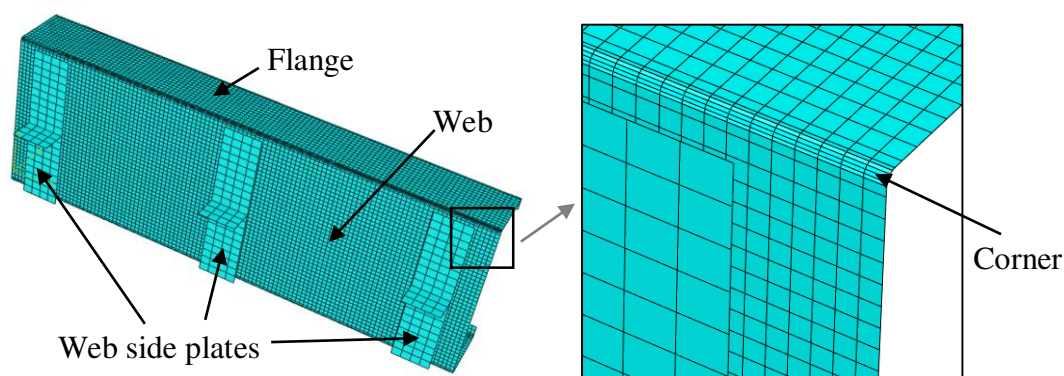
187 development of the FE models, the commercially available FE software package ABAQUS  
188 CAE 2017 was used.

189 When developing FE models single LCBs were employed considering the symmetry of the test  
190 setup instead of the back-to-back setup, in order to reduce the computational cost associated  
191 with simulation running time. Appropriate boundary conditions were introduced to simulate  
192 real conditions and LCBs were supported and loaded through the shear centre using single web  
193 side plates to reduce torsional effects. The contact between the web side plate and the LCB  
194 web was defined as tie constraints available in Abaqus. The shear centre location was  
195 calculated using THIN-WALL-2 [22] software. Similar FE models were employed previously  
196 [23], and deemed to provide good accuracy with test results.

### 197 3.2 Element type and FE mesh

198 S4R shell elements available in Abaqus were employed in the FE models as they account for  
199 finite membrane strains and large rotations, thus allowing for large-strain analysis [24]. S4R  
200 shell element type has four nodes and six degrees of freedom per each node. The successful  
201 use of S4R shell elements in thin-walled sections subject to shear has been previously proven  
202 by Sonu and Singh [25]. By conducting a mesh sensitivity analysis, it was found that 5 mm ×  
203 5 mm sized mesh was able to provide convergence with reasonably good accuracy. However,  
204 for the corners relatively smaller mesh of 1 mm was employed in the transverse direction to  
205 define the curvature. Relatively larger mesh was employed for the web side plates since more  
206 focus has been given to the LCB section behaviour, thus allowing more efficient simulation  
207 time. Figure 7 shows the FE mesh of the LCB section and web side plates.

208



209 Figure 7: FE mesh of LCB and web side plates.

210

### 211 3.3 Material model

212 Stainless steel exhibits non-linear stress-strain behaviour due to its pronounced strain  
213 hardening effect. Over the past years, numerous material models have been proposed to  
214 accurately capture this non-linear behaviour. Recently, the existing two-stage Ramberg-  
215 Osgood material model has been modified by Arrayago et al. [26] and the proposals are  
216 recommended to be included in future revisions of EN1993-1-4 [16]. Therefore to define the  
217 stress-strain relationship of stainless steel, modified two-stage Ramberg-Osgood material  
218 model proposed by Arrayago et al. [26] was used. Then true stress ( $\sigma_{true}$ ) and log plastic strain  
219 ( $\varepsilon_{ln}^{pl}$ ) were calculated using Eqs. (1) and (2) and incorporated into Abaqus as a multilinear curve  
220 with sufficient points to represent the accurate stress-strain behaviour. It is worth to note that  
221 for the developed cold-formed steel FE models, elastic, perfectly-plastic material model was  
222 incorporated.

$$223 \sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom}) \quad (1)$$

$$224 \varepsilon_{ln}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (2)$$

225 where  $\sigma_{nom}$  and  $\varepsilon_{nom}$  are engineering stress and strain, respectively.

### 226 3.4 Corner strength enhancement and residual stresses

227 During the cold-forming process of LCBs corner regions undergo larger plastic deformations.  
228 This results in considerable increase in material strength particularly in stainless steel which is  
229 termed as cold-working. Therefore, this strength enhancement is required to be considered in  
230 the FE modelling explicitly. Previous studies have been conducted to predict the strength  
231 enhancement due to the cold-working in stainless steel by Ashraf et al. [27] and Cruise and  
232 Gardner [28]. These proposed expressions were used to determine the corner material  
233 properties of stainless steel. In the FE modelling, these strength enhancement was introduced  
234 to the corner regions as mentioned in Cruise and Gardner [28]. To determine the corner 0.2%  
235 proof stress ( $\sigma_{0.2,c}$ ) and corner ultimate stress ( $\sigma_{u,c}$ ) Eqs. (3) [28] and (4) [27] were adopted,  
236 respectively. However, the effect of the residual stresses were not taken into account when  
237 developing FE models, as it has a negligible effect on the section capacity [5,29].

$$238 \sigma_{0.2,c} = \frac{1.673\sigma_{0.2,v}}{\left(\frac{r_i}{t}\right)^{0.126}} \quad (3)$$

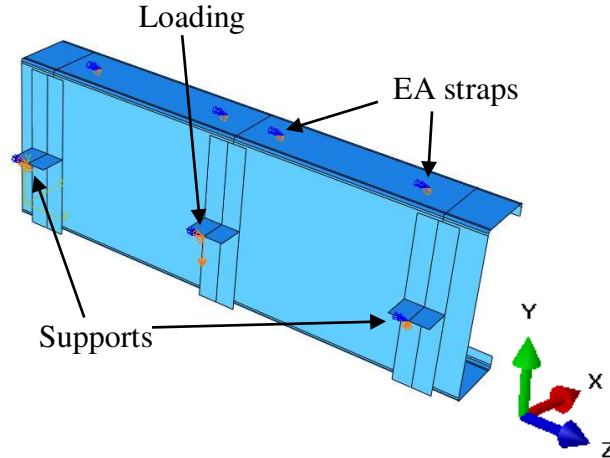
239 
$$\sigma_{u,c} = 0.75\sigma_{0.2,c} \left( \frac{\sigma_{u,v}}{\sigma_{0.2,v}} \right) \quad (4)$$

240 where  $\sigma_{0.2,v}$  and  $\sigma_{u,v}$  are 0.2% proof stress and ultimate stress of virgin material, respectively.

241 3.5 Loading and boundary conditions

242 In the experiments simply supported boundary conditions were maintained at the two supports.  
 243 Thus, a pin and a roller were assigned at the two supports of the beam in the FE models. Also,  
 244 the rotational degree of freedom about the longitudinal axis (z-axis) of the section was  
 245 restrained at these two supports to avoid any torsional effect. Suitable boundary conditions  
 246 were assigned to the flanges at the strap locations to simulate the effect of equal angle straps.  
 247 The mid-span loading was represented by assigning a vertical displacement to the section at  
 248 the mid-span and restraining suitable degrees of freedom. Figure 8 illustrates the assigned  
 249 boundary conditions in the FE models and Table 4 gives the details of boundary conditions  
 250 used. Note that in Table 4,  $u_x$ ,  $u_y$  and  $u_z$  are translations and  $\theta_x$ ,  $\theta_y$  and  $\theta_z$  are rotations in the  
 251 x, y and z directions, respectively while 0 denotes free and 1 denotes restrained conditions.

252



253 Figure 8: Boundary conditions assigned to the FE models.

254

255

256

257

258

259

Table 4: Boundary conditions used in the FE models.

	$u_x$	$u_y$	$u_z$	$\theta_x$	$\theta_y$	$\theta_z$
Left support	1	1	1	0	0	1
Right support	1	1	0	0	0	1
Mid span loading point	1	0	1	0	0	1
Strap locations	1	0	0	0	0	1

260

## 261 3.6 Geometric imperfections

262 Inclusion of geometric imperfections of thin-walled structures in FE modelling is important as  
 263 these geometric imperfections can massively alter the structural behaviour of thin-walled  
 264 structures. A study done by Schafer and Pekoz [30] suggested guidelines on treating these  
 265 geometric imperfections in numerical modelling. The effect of geometric imperfections to the  
 266 non-linear FE models was introduced by following the steps mentioned in [31]. Critical elastic  
 267 buckling mode shape was taken as the imperfection pattern of each FE model. In order to define  
 268 the imperfection amplitude ( $\omega_0$ ) in the FE modelling modified Dawson and Walker model  
 269 [32,33], as given by Eq. (5) was used. This has been previously used by many researchers in  
 270 the numerical modelling of stainless steel sections [25,34–36].

$$271 \quad \omega_0 = 0.023 \left( \frac{\sigma_{0.2}}{\sigma_{cr}} \right) t \quad (5)$$

272 where  $\sigma_{cr}$  is the critical elastic buckling stress of the most slender element of the section.

## 273 3.7 Analysis methods

274 Two analysis types were employed in the current study. For the inclusion of geometric  
 275 imperfection patterns in the non-linear FE analysis, a bifurcation buckling analysis was initially  
 276 performed and critical elastic buckling mode shapes were identified. Then, the imperfections  
 277 were introduced to the non-linear FE models. Thereafter, a non-linear static analysis was  
 278 employed on the FE models to study the shear behaviour of LCBs up to failure.

279 **4 Validation**

280 Tables 5 and 6 summarise the comparison of experimental and FE ultimate loads ( $V_{Exp}$ . and  
 281  $V_{FE}$ ) of stainless steel and cold-formed steel sections in shear, respectively. The purpose of  
 282 developing cold-formed steel FE models is discussed later in the Section 5. From the results, it  
 283 can be seen that developed FE models are able to predict the shear capacity of LCBs with a  
 284 reasonably good accuracy. The mean and coefficient of variance (COV) of the test to FE shear  
 285 capacity ratio are 0.99 and 0.070, respectively for the stainless steel sections and 0.99 and  
 286 0.084, respectively for the cold-formed steel sections.

287

288 Table 5: Comparison of experimental shear capacities with FE results and current design  
 289 provisions for stainless steel sections.

<i>LCB section</i>	$V_{FE}$ (kN)	$V_{Exp}/V_{FE}$	$V_{Exp}/V_{EN1993-1-4}$	$V_{Exp}/V_{DSM}$
LCB 100×50×15×1.2	16.9	1.09	1.18	1.06
LCB 100×50×15×1.5	24.3	1.01	1.06	1.10
LCB 100×50×15×2.0	35.0	1.03	1.08	1.24
LCB 150×65×15×1.2	20.8	1.04	1.13	1.05
LCB 150×65×15×1.5	29.4	0.89	0.95	0.86
LCB 150×65×15×2.0	42.8	1.02	1.04	0.98
LCB 200×75×15×1.2	23.8	0.97	1.07	1.02
LCB 200×75×15×2.0	52.5	0.90	0.96	0.87
Mean		0.99	1.06	1.02
COV		0.070	0.072	0.122

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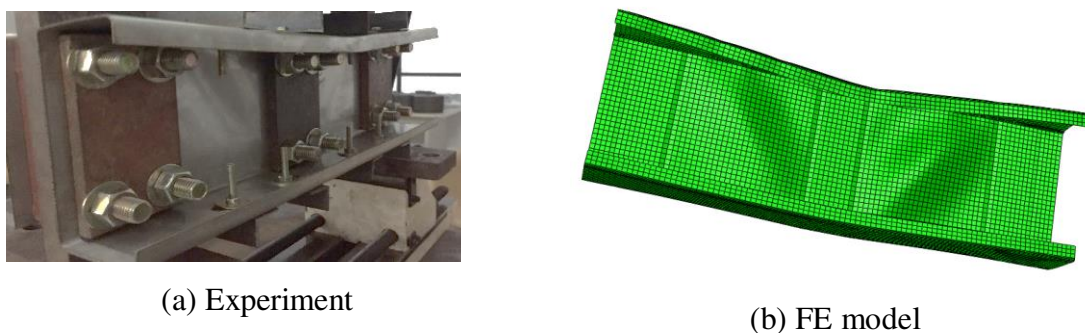
296 Table 6: Comparison of experimental [12] and FE shear capacities for cold-formed steel  
 297 sections.

<i>LCB section</i>	$V_{Exp.}$ (kN)	$V_{FE}$ (kN)	$V_{Exp.}/V_{FE}$
LCB 120×50×18×1.5	43.3	47.8	0.91
LCB 120×50×18×2.0	38.1	34.9	1.09
LCB 160×65×15×1.5	54.5	55.2	0.99
LCB 160×65×15×2.0	73.8	77.6	0.95
LCB 200×75×15×1.5	57.0	61.9	0.92
LCB 200×75×15×2.0	55.1	50.1	1.10
Mean			0.99
COV			0.084

298

299 In order to demonstrate the ability of the developed FE models to capture the shear failure  
 300 modes of LCBs, failure modes of stainless steel tests and FE models were compared. Figures  
 301 9 and 10 illustrate these comparisons of shear failure modes as captured during the experiment  
 302 and from the FE model. It can be concluded that the developed FE models were able to capture  
 303 the failure modes in a fairly similar manner. Further, experimental and FE load-deflection  
 304 curves for LCB 150×65×15×2.0 are compared in Figure 11. Due to the slip at the bolt  
 305 connections experimental curve exhibits higher deflections compared to FE curve.

306



307 Figure 9: Shear failure mode of LCB 150×65×15×2.0 section.

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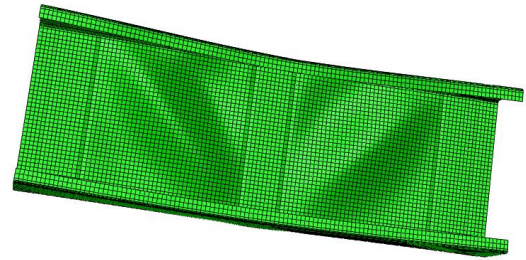
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(a) Experiment

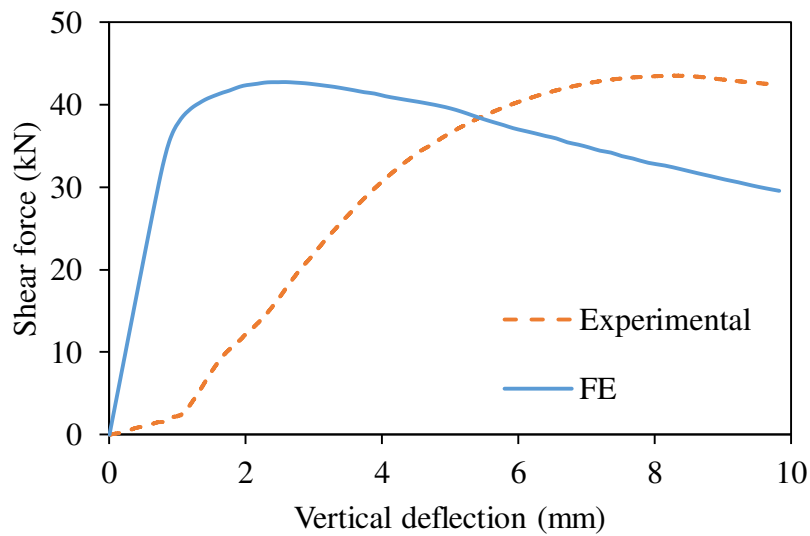


(b) FE model

Figure 10: Shear failure mode of LCB 200×75×15×1.2 section.

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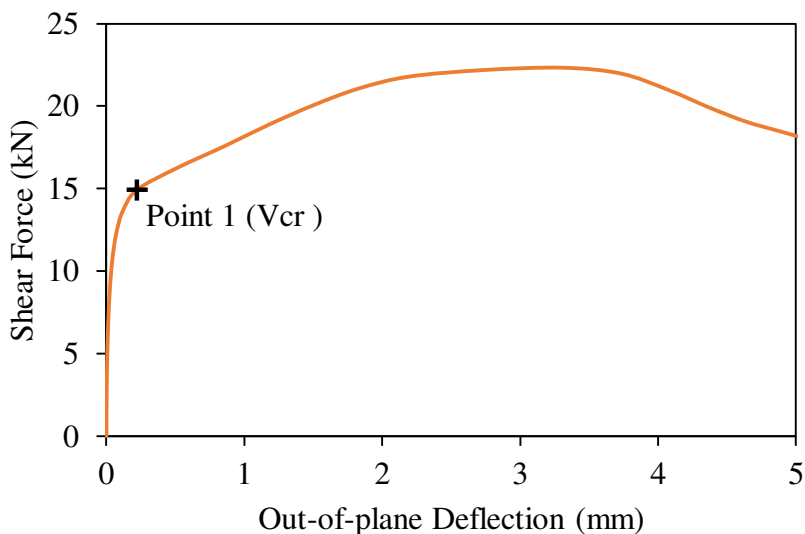
Figure 11: Comparison of experimental and FE load-deflection curves for LCB 150×65×15×2.0 section.

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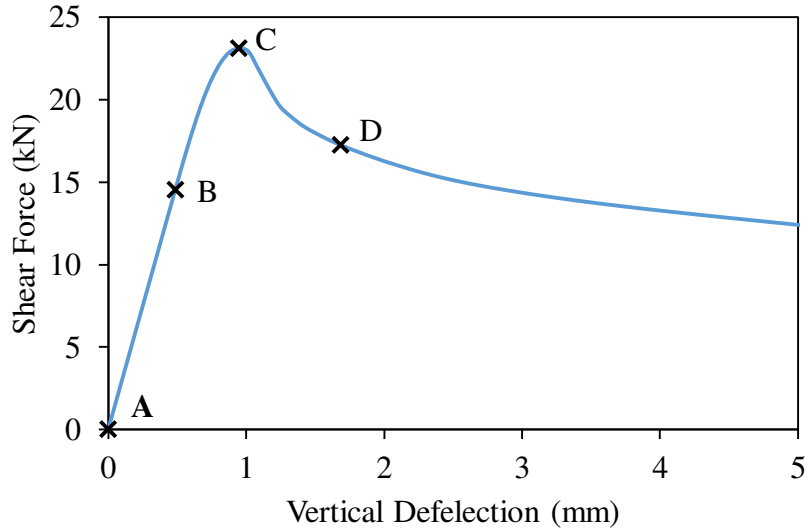
317 Figure 12 illustrates the FE load-deflection curve for stainless steel LCB 200×75×15×1.2  
 318 section obtained without the effect of imperfections. For the FE model without imperfections,  
 319 a negligible imperfection amplitude ( $\omega_0/1000$ ) was introduced so that there was no  
 320 considerable effect from the imperfections on the shear behaviour of the section. The lateral  
 321 deflection of the mid-point of one span (out-of-plane deflection of the web) was monitored  
 322 against the shear force. Out-of-plane deflection of the web began at Point 1 as it can be seen  
 323 from the load-deflection curve. Thus, shear force at Point 1 can be taken as the elastic shear  
 324 buckling load of LCB 200×75×15×1.2 section. Therefore, from Figure 12 elastic shear  
 325 buckling load of LCB 200×75×15×1.2 section can be taken as 14.92 kN. Using Eq. (18), elastic

326 shear buckling load of the same section was also calculated and is equal to 15.21 kN, thus  
327 demonstrating the ability of developed FE models to capture elastic shear buckling load.

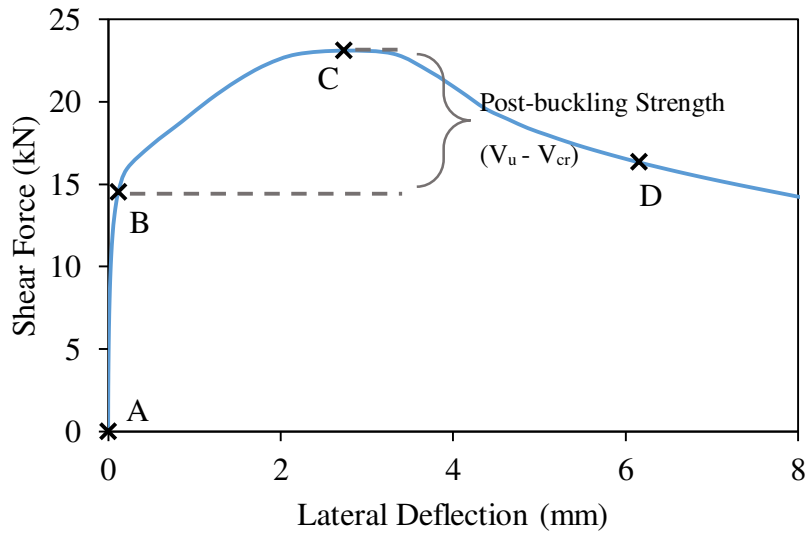


328  
329 Figure 12: Load-deflection curve for LCB 200×75×15×1.2 section without imperfections.  
330

331 Figure 13 illustrates the load-vertical deflection curve against the load-lateral deflection curve  
332 for the stainless steel LCB 200×75×15×1.2 section without imperfections. It can be seen that  
333 there is a considerable amount of post-buckling strength in this section when the web started  
334 to buckle out-of-plane at Point B and then reached its shear capacity at point C. The existence  
335 of this considerable amount of post-buckling strength particularly in slender LCB sections  
336 under shear was also highlighted by Keerthan and Mahendran [12]. Figure 14 shows the FE  
337 deformation modes and stress patterns of LCB 200×75×15×1.2 section under progressive  
338 loading at points A, B, C and D where these points represent initial, buckling, peak and post-  
339 peak conditions, respectively.



(a) Load-vertical deflection curve



(b) Load-lateral deflection curve

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Figure 13: Load-deflection curves and post-buckling strength for LCB 200×75×15×1.2 section.

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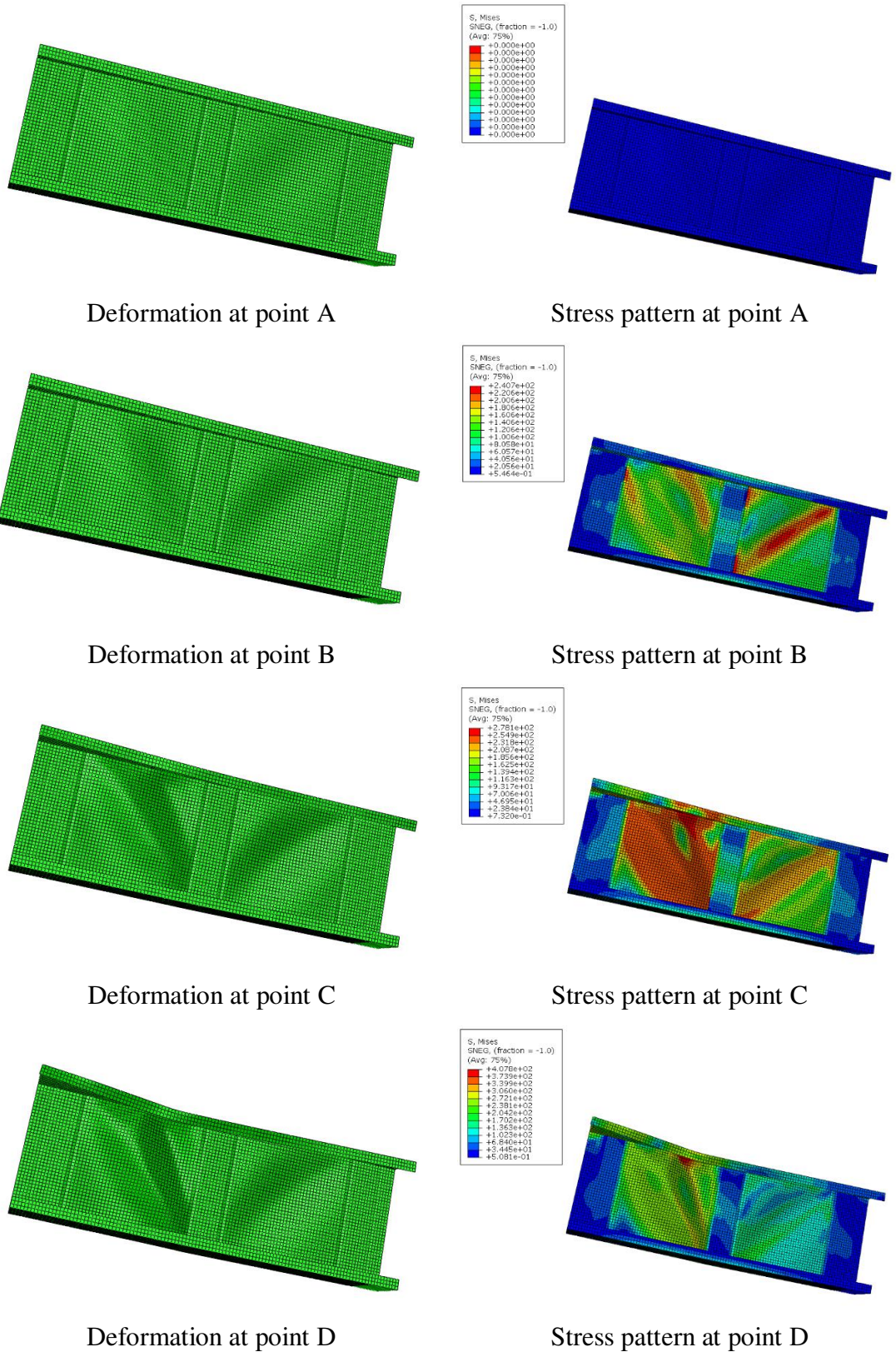
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349 Figure 14: FE deformation modes of LCB 200x75x15x1.2 section under progressive loading.

350

351 **5 Parametric study**

352 A detailed parametric study was conducted in view of gathering comprehensive database to  
353 investigate the shear behaviour of stainless steel LCBs. 100 shear FE models of stainless steel  
354 LCBs were developed at this stage. For the parametric study, five different stainless steel  
355 grades from EN1993-1-4 [16] including both austenitic and duplex grades were considered.  
356 Four different LCB sections and five different thicknesses were selected in order to cover a  
357 wide range of slenderness values. The parameters used for the study are summarised in Table  
358 7. The aspect ratio ( $a/d_1$ ) of 1.0 was used for all FE models developed here. Young's modulus  
359 and Poisson's ratio of all stainless steel grades used in the parametric study were taken as  
360 200,000 MPa and 0.3, respectively according to EN1993-1-4 [16]. Tables 8-12 summarise the  
361 parametric study results of ultimate shear capacities of cold-formed stainless steel LCBs.

362

363 Table 7: Summary of parameters used in the parametric study.

<i>Section</i>	<i>t (mm)</i>	<i>Stainless steel grade</i>	<i>No. of FE models</i>
LCB 100×50×15xt	1, 2, 3, 4, 5	Austenitic- 1.4301,	25
LCB 150×65×15xt		1.4311, 1.4318	25
LCB 200×75×15xt		Duplex- 1.4362,	25
LCB 250×75×15xt		1.4462	25
Total			100

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Table 8: Comparison of parametric study results for stainless steel grade 1.4301.

<i>Section</i>	$V_{FE}$ (kN)	$V_{FE}/V_{EN1993-1-4}$	$V_{FE}/V_{EN1993-1-4}$ <i>Proposed</i>	$V_{FE}/V_{DSM}$	$V_{FE}/V_{DSM}$ <i>Proposed</i>
Stainless steel grade					
1.4301					
LCB 100×50×15×1	12.93	1.13	1.11	1.01	1.12
LCB 100×50×15×2	31.37	1.03	1.01	1.18	1.01
LCB 100×50×15×3	56.21	1.25	1.03	1.44	1.03
LCB 100×50×15×4	86.84	1.48	1.06	1.71	1.06
LCB 100×50×15×5	125	1.74	1.13	2.01	1.12
LCB 150×65×15×1	15.28	1.11	1.06	1.04	1.09
LCB 150×65×15×2	39.66	1.02	1.01	0.98	1.01
LCB 150×65×15×3	67.51	0.98	0.97	1.13	0.97
LCB 150×65×15×4	104.21	1.15	1.00	1.33	1.00
LCB 150×65×15×5	143.14	1.28	1.00	1.48	1.00
LCB 200×75×15×1	17.17	1.12	1.05	1.08	1.08
LCB 200×75×15×2	47.90	1.05	1.02	0.94	1.04
LCB 200×75×15×3	81.99	0.94	0.99	1.02	1.00
LCB 200×75×15×4	123.32	1.01	1.00	1.16	1.00
LCB 200×75×15×5	166.53	1.10	0.98	1.27	0.98
LCB 250×75×15×1	18.44	1.12	1.04	1.08	1.05
LCB 250×75×15×2	53.16	1.04	1.01	0.96	1.03
LCB 250×75×15×3	92.38	0.99	0.98	0.91	0.99
LCB 250×75×15×4	138.29	0.90	0.98	1.04	0.98
LCB 250×75×15×5	187.86	0.98	0.97	1.13	0.97
Mean		1.12	1.02	1.20	1.03
COV		0.177	0.042	0.234	0.046

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Table 9: Comparison of parametric study results for stainless steel grade 1.4311.

<i>Section</i>	$V_{FE}$ (kN)	$V_{FE}/V_{EN1993-1-4}$	$V_{FE}/V_{EN1993-1-4}$ <i>Proposed</i>	$V_{FE}/V_{DSM}$	$V_{FE}/V_{DSM}$ <i>Proposed</i>
Stainless steel grade					
1.4311					
LCB 100×50×15×1	15.47	1.14	1.10	1.04	1.13
LCB 100×50×15×2	38.14	0.99	1.03	1.14	1.03
LCB 100×50×15×3	65.23	1.15	0.99	1.33	0.99
LCB 100×50×15×4	96.47	1.30	0.99	1.51	0.98
LCB 100×50×15×5	137.98	1.53	1.04	1.76	1.03
LCB 150×65×15×1	17.89	1.10	1.05	1.05	1.08
LCB 150×65×15×2	48.6	1.04	1.03	0.96	1.03
LCB 150×65×15×3	81.51	0.94	0.98	1.08	0.98
LCB 150×65×15×4	123.1	1.08	0.98	1.25	0.98
LCB 150×65×15×5	167.28	1.19	0.98	1.37	0.98
LCB 200×75×15×1	19.98	1.11	1.04	1.08	1.06
LCB 200×75×15×2	57.48	1.06	1.03	0.96	1.05
LCB 200×75×15×3	100.71	1.02	1.02	0.99	1.02
LCB 200×75×15×4	146.33	0.95	0.99	1.10	0.99
LCB 200×75×15×5	197.48	1.03	0.97	1.19	0.97
LCB 250×75×15×1	20.99	1.10	1.01	1.07	1.02
LCB 250×75×15×2	63.04	1.05	1.00	0.98	1.03
LCB 250×75×15×3	113.23	1.02	1.00	0.89	1.01
LCB 250×75×15×4	168.89	0.97	1.00	1.00	1.00
LCB 250×75×15×5	225.49	0.94	0.97	1.08	0.97
Mean		1.09	1.01	1.14	1.02
COV		0.127	0.033	0.186	0.039

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Table 10: Comparison of parametric study results for stainless steel grade 1.4318.

<i>Section</i>	$V_{FE}$ (kN)	$V_{FE}/V_{EN1993-1-4}$	$V_{FE}/V_{EN1993-1-4}$ <i>Proposed</i>	$V_{FE}/V_{DSM}$	$V_{FE}/V_{DSM}$ <i>Proposed</i>
Stainless steel grade 1.4318					
LCB 100×50×15×1	17.62	1.13	1.09	1.04	1.12
LCB 100×50×15×2	44.83	0.96	1.04	1.11	1.05
LCB 100×50×15×3	76.98	1.13	1.01	1.30	1.01
LCB 100×50×15×4	113.57	1.27	1.00	1.47	1.00
LCB 100×50×15×5	161.41	1.48	1.05	1.71	1.04
LCB 150×65×15×1	20.26	1.10	1.04	1.05	1.07
LCB 150×65×15×2	56.74	1.05	1.03	0.93	1.04
LCB 150×65×15×3	96	0.92	0.99	1.06	0.99
LCB 150×65×15×4	145.37	1.06	1.00	1.22	1.00
LCB 150×65×15×5	196.74	1.16	0.99	1.34	0.99
LCB 200×75×15×1	22.33	1.10	1.02	1.07	1.04
LCB 200×75×15×2	66.36	1.06	1.03	0.98	1.05
LCB 200×75×15×3	117.87	1.04	1.02	0.96	1.03
LCB 200×75×15×4	173.29	0.93	1.01	1.07	1.01
LCB 200×75×15×5	234.13	1.02	0.99	1.17	1.00
LCB 250×75×15×1	23.48	1.09	1.00	1.06	1.00
LCB 250×75×15×2	72.99	1.06	1.01	1.00	1.04
LCB 250×75×15×3	132.79	1.04	1.01	0.93	1.03
LCB 250×75×15×4	196.61	1.01	1.00	0.97	1.00
LCB 250×75×15×5	267.77	0.92	1.00	1.06	1.00
Mean		1.08	1.02	1.13	1.03
COV		0.119	0.024	0.175	0.030

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Table 11: Comparison of parametric study results for stainless steel grade 1.4362.

<i>Section</i>	$V_{FE}$ (kN)	$V_{FE}/V_{EN1993-1-4}$	$V_{FE}/V_{EN1993-1-4}$ <i>Proposed</i>	$V_{FE}/V_{DSM}$	$V_{FE}/V_{DSM}$ <i>Proposed</i>
Stainless steel grade 1.4362					
LCB 100×50×15×1	20.63	1.10	1.06	1.03	1.09
LCB 100×50×15×2	53.96	0.99	1.03	1.04	1.03
LCB 100×50×15×3	89.5	1.02	0.97	1.18	0.97
LCB 100×50×15×4	129.49	1.13	0.94	1.30	0.94
LCB 100×50×15×5	174.25	1.24	0.93	1.43	0.93
LCB 150×65×15×1	23.8	1.09	1.02	1.05	1.05
LCB 150×65×15×2	70.09	1.08	1.05	0.97	1.06
LCB 150×65×15×3	116.54	0.95	0.99	1.00	0.99
LCB 150×65×15×4	171.32	0.97	0.97	1.12	0.97
LCB 150×65×15×5	226.62	1.04	0.94	1.20	0.94
LCB 200×75×15×1	25.71	1.08	1.00	1.05	1.01
LCB 200×75×15×2	79.53	1.06	1.02	0.99	1.05
LCB 200×75×15×3	146.65	1.06	1.04	0.93	1.05
LCB 200×75×15×4	212.11	0.97	1.01	1.02	1.02
LCB 200×75×15×5	277.55	0.94	0.97	1.08	0.97
LCB 250×75×15×1	26.4	1.05	0.96	1.02	0.95
LCB 250×75×15×2	87.15	1.06	1.01	1.02	1.04
LCB 250×75×15×3	159.6	1.03	1.00	0.94	1.02
LCB 250×75×15×4	245.71	1.04	1.02	0.94	1.03
LCB 250×75×15×5	321.48	0.94	0.98	0.99	0.99
Mean		1.04	0.99	1.07	1.00
COV		0.070	0.038	0.119	0.047

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Table 12: Comparison of parametric study results for stainless steel grade 1.4462.

<i>Section</i>	$V_{FE}$ (kN)	$V_{FE}/V_{EN1993-1-4}$	$V_{FE}/V_{EN1993-1-4}$ <i>Proposed</i>	$V_{FE}/V_{DSM}$	$V_{FE}/V_{DSM}$ <i>Proposed</i>
Stainless steel grade 1.4462					
LCB 100×50×15×1	22.32	1.11	1.06	1.04	1.09
LCB 100×50×15×2	58.97	1.03	1.04	1.02	1.04
LCB 100×50×15×3	98.09	1.00	0.98	1.16	0.98
LCB 100×50×15×4	140.72	1.10	0.94	1.27	0.94
LCB 100×50×15×5	188.3	1.21	0.92	1.39	0.92
LCB 150×65×15×1	25.52	1.09	1.02	1.05	1.04
LCB 150×65×15×2	75.59	1.07	1.04	0.97	1.06
LCB 150×65×15×3	127.68	0.99	1.00	0.99	1.00
LCB 150×65×15×4	186.87	0.95	0.97	1.10	0.98
LCB 150×65×15×5	245.93	1.01	0.94	1.17	0.94
LCB 200×75×15×1	27.1	1.06	0.98	1.04	0.99
LCB 200×75×15×2	86.63	1.08	1.03	1.01	1.06
LCB 200×75×15×3	160.23	1.07	1.05	0.96	1.06
LCB 200×75×15×4	230.56	1.00	1.01	1.00	1.02
LCB 200×75×15×5	302.5	0.92	0.97	1.06	0.97
LCB 250×75×15×1	27.55	1.02	0.94	0.99	0.92
LCB 250×75×15×2	93.61	1.06	1.00	1.02	1.03
LCB 250×75×15×3	174.2	1.04	1.01	0.96	1.03
LCB 250×75×15×4	267.71	1.04	1.02	0.92	1.04
LCB 250×75×15×5	356.07	0.99	1.00	0.99	1.00
Mean		1.04	1.00	1.06	1.01
COV		0.060	0.040	0.109	0.049

395 Due to the pronounced strain hardening effect of stainless steel, significant strength increment  
396 can be envisaged beyond the yield strength of the material which is conventionally taken as the  
397 0.2 % proof stress. In order to highlight this strain hardening effect on the shear behaviour of  
398 stainless steel LCBs, further analysis was conducted. Both compact sections and slender

399 sections were considered here. The shear capacities of twenty LCBs of stainless steel grade  
400 1.4301 was compared with the results obtained from cold-formed LCBs. The validated cold-  
401 formed steel FE models were incorporated for this purpose. When developing cold-formed  
402 steel FE models stress was limited to the yield stress of grade 1.4301. Table 13 summarises the  
403 shear capacities and percentage increment of strength for each section where  $\bar{\lambda}_w$  is the web  
404 slenderness calculated from EN1993-1-4 [16].

405

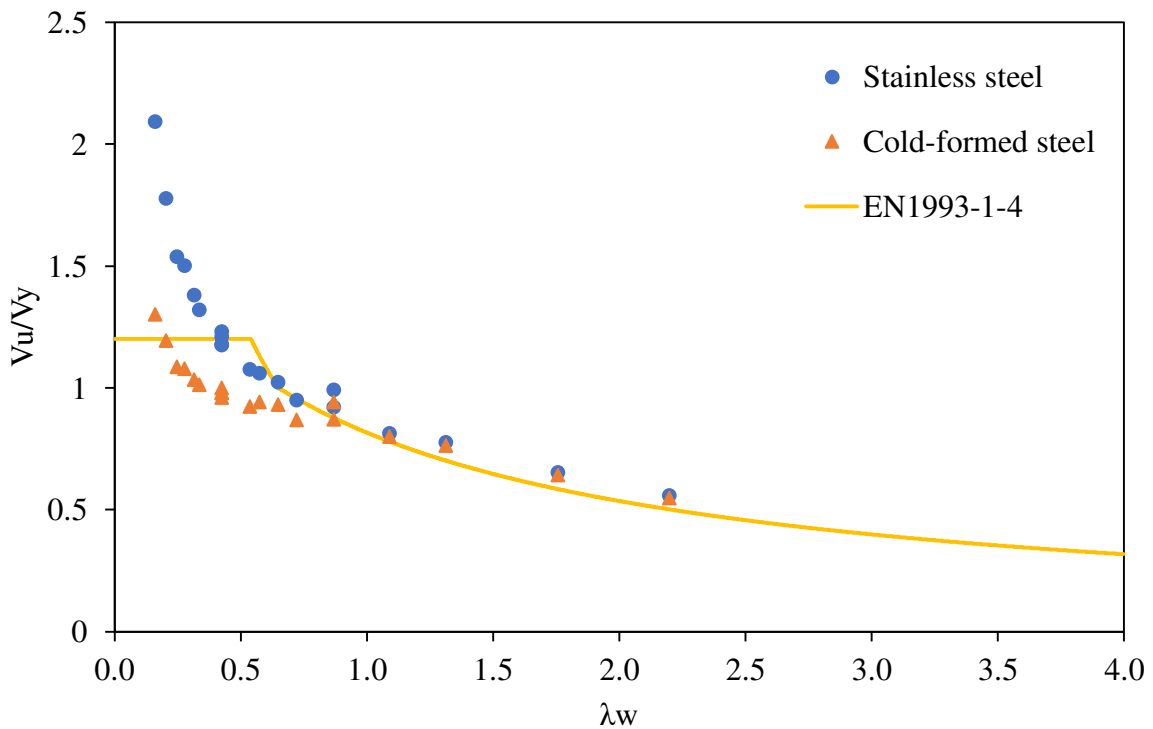
406 Table 13: Comparison of shear capacities of stainless steel and cold-formed steel LCBs.

<i>Section</i>	<i>t</i> (mm)	<i>d<sub>l</sub>/t</i>	$\bar{\lambda}_w$	<i>V<sub>stainless steel</sub></i> (kN)	<i>V<sub>cold-formed steel</sub></i> (kN)	% <i>Increment</i>
LCB 100×50×15×1	1.0	98.0	0.87	12.93	12.28	5.29
LCB 100×50×15×2	2.0	48.0	0.43	31.37	25.51	22.97
LCB 100×50×15×3	3.0	31.3	0.28	56.21	40.39	39.17
LCB 100×50×15×4	4.0	23.0	0.20	86.84	58.38	48.75
LCB 100×50×15×5	5.0	18.0	0.16	125	77.89	60.48
LCB 150×65×15×1	1.0	148.0	1.31	15.28	15.01	1.80
LCB 150×65×15×2	2.0	73.0	0.65	39.66	36.13	9.77
LCB 150×65×15×3	3.0	48.0	0.43	67.51	56.13	20.27
LCB 150×65×15×4	4.0	35.5	0.31	104.21	77.94	33.71
LCB 150×65×15×5	5.0	28.0	0.25	143.14	101.04	41.67
LCB 200×75×15×1	1.0	198.0	1.76	17.17	16.92	1.48
LCB 200×75×15×2	2.0	98.0	0.87	47.9	45.43	5.44
LCB 200×75×15×3	3.0	64.7	0.57	81.99	72.74	12.72
LCB 200×75×15×4	4.0	48.0	0.43	123.32	100.01	23.31
LCB 200×75×15×5	5.0	38.0	0.34	166.53	127.82	30.28
LCB 250×75×15×1	1.0	248.0	2.20	18.44	18.08	1.99
LCB 250×75×15×2	2.0	123.0	1.09	53.16	52.27	1.70
LCB 250×75×15×3	3.0	81.3	0.72	92.38	84.44	9.40
LCB 250×75×15×4	4.0	60.5	0.54	138.29	118.78	16.43
LCB 250×75×15×5	5.0	48.0	0.43	187.86	152.99	22.79

407

408 According to Table 13, it can be seen that when  $d_1/t_w$  ratio is less than 28, more than 40 % shear  
 409 capacity increment can be expected in stainless steel sections due to the effect of strain  
 410 hardening. Therefore, it is concluded that this strain hardening effect is more pronounced in  
 411 compact sections while that for slender sections is negligible. This strength increment existing  
 412 in compact sections is known as inelastic reserve capacity. Similar inelastic reserve capacities  
 413 were observed for the stainless steel rectangular hollow sections in shear by Sonu and Singh  
 414 [25]. This effect is further highlighted for stainless steel angles and channels in bending by  
 415 Theofanous et al. [4]. This shear capacity increment in compact sections due to the strain  
 416 hardening of stainless steel is further highlighted from Figure 15 where sectional shear capacity  
 417 ( $V_u$ ) to yield load ( $V_y$ ) ratio was compared with the web slenderness ( $\bar{\lambda}_w$ ).

418



419 Figure 15: Comparison of shear capacities of stainless steel and cold-formed steel LCBs.

420

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424 **6 Current shear design rules**

425 6.1 EN1993-1-4 [16] shear design rules

426 EN1993-1-4 [16] is based on the effective width method where traditional cross-section  
 427 classification approach is used to divide cross sections into different behavioural classes by  
 428 assuming the class of its most slender element while incorporating the effect of material  
 429 properties, support conditions and loading patterns. Design provisions for shear introduced in  
 430 EN1993-1-4 [16] are to be referred alongside with the provisions provided in EN1993-1-1 [37]  
 431 and EN1993-1-5 [17]. Eq. (6) has been introduced in EN1993-1-4 [16] to calculate the sectional  
 432 shear resistance ( $V_{b,Rd}$ ) which is taken as the sum of web shear resistance ( $V_{bw,Rd}$ ) and flange  
 433 shear resistance ( $V_{bf,Rd}$ ).

434 
$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (6)$$

435 where,  $f_{yw}$  is the web yield stress,  $h_w$  is the clear web depth between flanges and  $t_w$  is the web  
 436 thickness. Here  $\eta = 1.2$  is recommended and  $\gamma_{M1}$  is a partial factor [16] .

437 Eq. (7) gives the web shear resistance,  $V_{bw,Rd}$  where  $\chi_w$  is the web shear buckling reduction  
 438 factor, values for which for webs with rigid end-post are given in Table 14.

439 
$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (7)$$

440

441 Table 14: Web shear buckling reduction factor,  $\chi_w$  for webs with rigid end post according to  
 442 EN1993-1-4 [16].

$\chi_w$	
$\bar{\lambda}_w \leq 0.65/\eta$	$\eta$
$0.65/\eta < \bar{\lambda}_w < 0.65$	$0.65/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 0.65$	$1.56/(0.91 + \bar{\lambda}_w)$

443

444 In Table 14,  $\bar{\lambda}_w$  is the web slenderness which is defined in Eq. (8) for webs with transverse  
 445 stiffeners at supports and mid span where  $\varepsilon$  and  $k_\tau$  are material factor and web shear buckling  
 446 coefficient, respectively.

447 
$$\bar{\lambda}_w = \frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}} \quad (8)$$

448 Material factor ( $\varepsilon$ ) is defined by Eq. (9) while web shear buckling coefficient ( $k_\tau$ ) for plates  
 449 with rigid transverse stiffeners and without longitudinal stiffeners is given by Eqs. (10) and  
 450 (11) where  $f_y$  is the yield stress,  $E$  is the modulus of elasticity and  $a$  is the distance between  
 451 transverse stiffeners.

452 
$$\varepsilon = \sqrt{\frac{235}{f_y} \frac{E}{210\,000}} \quad (9)$$

453 
$$k_\tau = 5.34 + \frac{4.00}{(a/h_w)^2} \text{ for } \frac{a}{h_w} \geq 1 \quad (10)$$

454 
$$k_\tau = 4.00 + \frac{5.34}{(a/h_w)^2} \text{ for } \frac{a}{h_w} < 1 \quad (11)$$

455 Flange shear resistance ( $V_{bf,Rd}$ ) given in Eq. (6) is defined by Eq. (12) where  $b_f$  and  $t_f$  are  
 456 flange width and thickness, respectively which provides the least axial resistance while  $f_{yf}$  is  
 457 the flange yield stress. Here  $c$  is given in Eq. (13). It is of note that Eq. (12) is only valid if  
 458  $M_{Ed} < M_{f,Rd}$  where  $M_{Ed}$  is the design bending moment and  $M_{f,Rd}$  is the effective flange  
 459 moment resistance.

460 
$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \left( 1 - \left( \frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) \quad (12)$$

461 
$$c = a \left[ 0.17 + \frac{3.5 b_f t_f^2 f_{yf}}{t_w h_w^2 f_{yw}} \right] \text{ and } \frac{c}{a} \leq 0.65 \quad (13)$$

## 462 6.2 Direct strength method (DSM)

463 The direct strength method is an alternative to the conventional effective width method [38].  
 464 The accurate member elastic stability is the fundamental theory on which DSM is formed  
 465 where the strength of a section is calculated considering all the elastic instabilities of the gross  
 466 cross section [39]. In DSM design resistance equations, the strength of a cross-section is  
 467 defined as a function of overall slenderness of the cross section ( $\lambda$ ). DSM shear capacity ( $V_v$ )  
 468 prediction equations proposed by Pham and Hancock [40] are given in Eqs. (14) and (15) where  
 469 two equations represent the shear yielding, and elastic and inelastic shear buckling regions,  
 470 respectively while  $\lambda$  is defined as in Eq. (16).

471 
$$\frac{V_v}{V_y} = 1 \text{ for } \lambda \leq 0.815 \quad (14)$$

472  $\frac{V_y}{V_{cr}} = \left[ 1 - 0.15 \left( \frac{1}{\lambda^2} \right)^{0.4} \right] \left( \frac{1}{\lambda^2} \right)^{0.4}$  for  $\lambda > 0.815$  (15)

473 where

474  $\lambda = \sqrt{\frac{V_y}{V_{cr}}}$  (16)

475 When calculating  $\lambda$ , shear yield capacity ( $V_y$ ) and elastic shear buckling capacity ( $V_{cr}$ ) are taken  
476 as defined by Eqs. (17) and (18), respectively.

477  $V_y = 0.6 f_{yw} d_1 t_w$  (17)

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

479 where  $f_{yw}$  is the web yield stress,  $d_1$  is the flat depth of the web,  $t_w$  is the web thickness,  $E$  is  
480 the Young's modulus and  $\nu$  is the Poisson's ratio. Here  $k$  is the shear buckling coefficient of  
481 the section. Keerthan and Mahendran [12] proposed a set of equations (Eqs. (19)-(23)) to  
482 calculate the shear buckling coefficient,  $k$  of LCBs considering the additional fixity available  
483 at the web-flange juncture of LCBs.

484  $k = k_{ss} + 0.23(k_{sf} - k_{ss})$  (19)

485  $k_{ss} = 5.34 + \frac{4}{(a/d_1)^2}$  for  $\frac{a}{d_1} \geq 1$  (20)

486  $k_{ss} = 4 + \frac{5.34}{(a/d_1)^2}$  for  $\frac{a}{d_1} < 1$  (21)

487  $k_{sf} = 8.98 + \frac{5.61}{(a/d_1)^2} - \frac{1.99}{(a/d_1)^3}$  for  $\frac{a}{d_1} \geq 1$  (22)

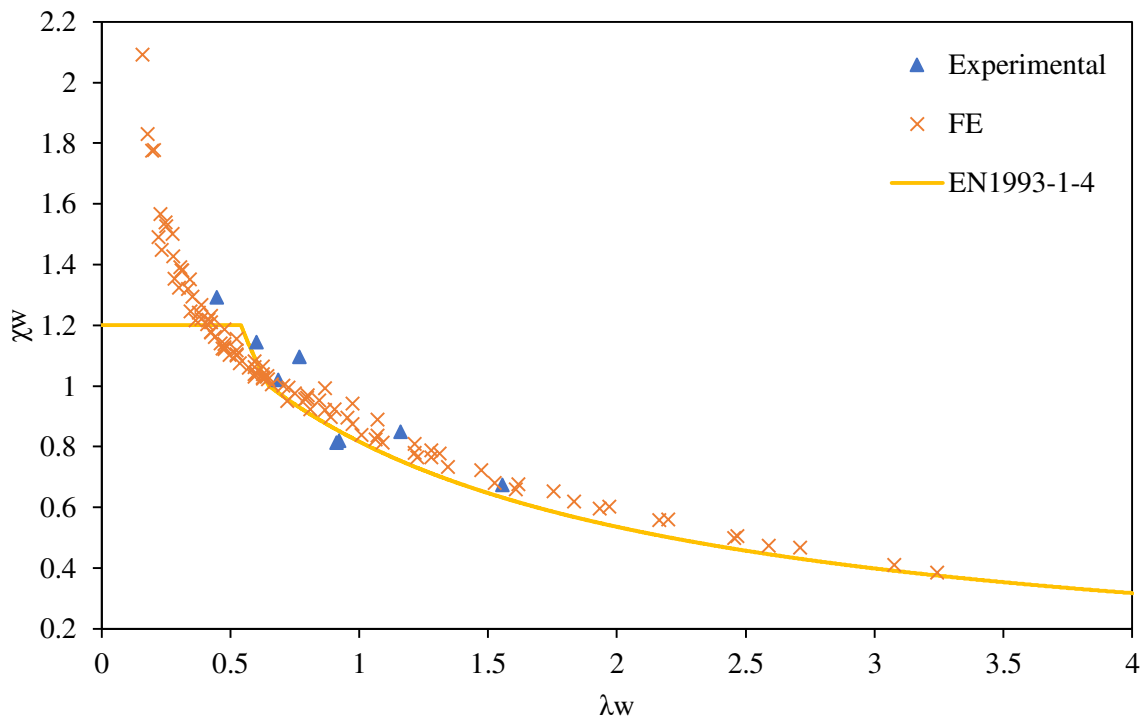
488  $k_{sf} = \frac{5.34}{(a/d_1)^2} + \frac{2.31}{(a/d_1)} - 3.44 + \frac{8.39}{(a/d_1)}$  for  $\frac{a}{d_1} < 1$  (23)

### 489 6.3 Performance of current design rules

490 The experimental results and developed shear FE models of stainless steel LCBs were utilised  
491 to assess the applicability of EN1993-1-4 [16] and DSM shear design rules described in the  
492 above sections. Table 5 includes the comparison of current shear design rules discussed here  
493 with the experimental results while Tables 8-12 compare the performance of the current  
494 EN1993-1-4 [16] and DSM shear design rules with the obtained FE results from the parametric  
495 study. The results show that experimental and FE shear capacities to predicted shear capacities

496 ratio has a mean and COV of 1.07 and 0.118, respectively for the current EN1993-1-4 [16]  
497 predictions while that for the current DSM predictions are 1.11 and 0.176, respectively.  
498 Moreover, Figures 16 and 17 illustrate the comparison of experimental and FE shear capacities  
499 with the current EN1993-1-4 [16] and DSM shear design curves, respectively. From both  
500 comparisons it is evident that the existing shear design rules are too conservative in particularly  
501 for compact sections.

502



503 Figure 16: Comparison of experimental and FE shear capacities with the current EN1993-1-4  
504 [16] shear capacity prediction curve.

505

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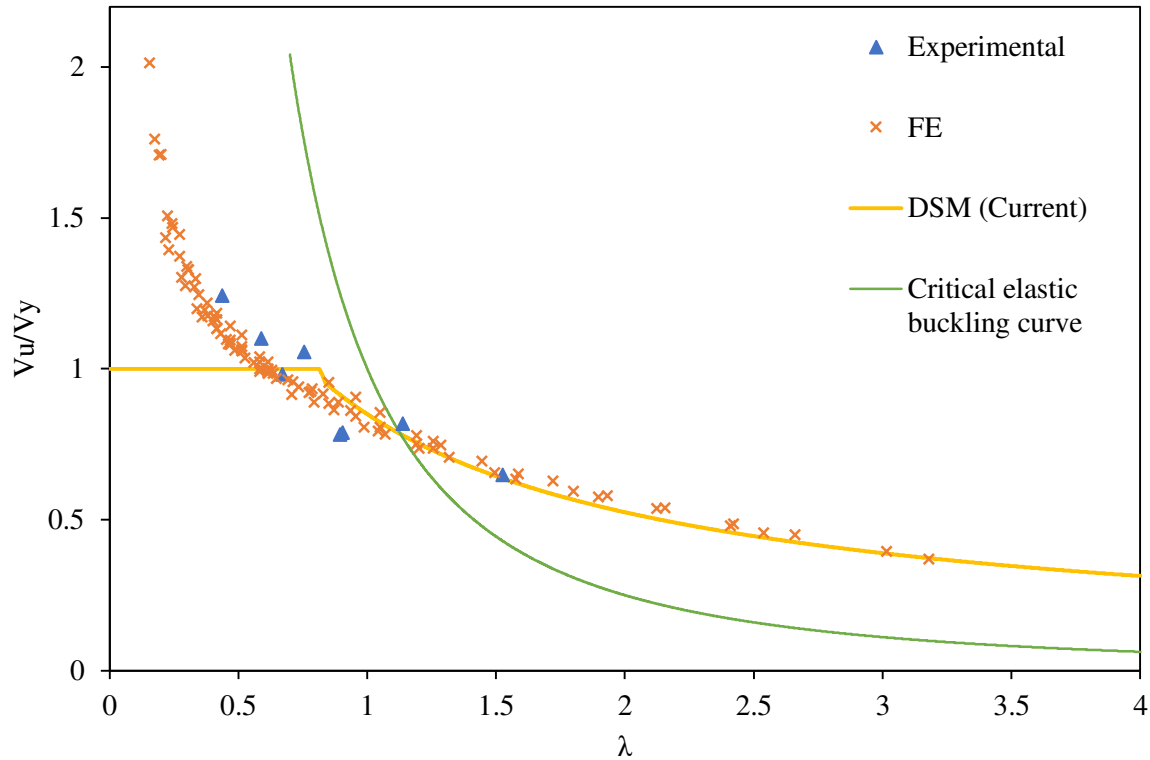
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512

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514





515 Figure 17: Comparison of experimental and FE shear capacities with the current DSM shear  
 516 capacity prediction curve.  
 517

## 518 7 Proposed shear design rules

519 In order to address the previously discussed shortcomings present in the current EN1993-1-4  
 520 [16] and DSM shear design provisions, attempts were made to modify the shear design rules  
 521 while aiming to improve the prediction accuracy. When developing such design rules, special  
 522 attention was given to capture the pronounced inelastic reserve capacity in compact stainless  
 523 steel LCBs in shear.

### 524 7.1 Proposed EN1993-1-4 [16] shear design rules

525 The applicability of the current shear design rules to predict shear behaviour of stainless steel  
 526 LCBs was evaluated and required modifications were made to Eqs. (6)-(13) given in Section  
 527 6.1 in view of enhancing the prediction accuracy. For the calculation of sectional shear  
 528 resistance ( $V_{b,Rd}$ ) using Eq. (6), the flange shear resistance ( $V_{bf,Rd}$ ) is required to be taken into  
 529 account as given by Eq. (12). Therefore, required modifications were made to the web shear  
 530 buckling coefficient ( $\chi_w$ ) in Eq. (7) by comparing the web shear resistance ( $V_{bw,Rd}$ ) with the

531 flange shear resistance reduced FE and experimental shear capacities ( $V_{FE} (& Exp.) - V_{bf,Rd}$ ).  
 532 However, it is worth to note that for almost all the sections studied, the condition  $M_{Ed} < M_{f,Rd}$   
 533 was not satisfied while for very few sections this condition was satisfied, but yet for those  
 534 sections flange shear resistance ( $V_{bf,Rd}$ ) was negligible.

535 A set of expressions were proposed for the web shear buckling coefficient ( $\chi_w$ ) as functions of  
 536 web slenderness ( $\bar{\lambda}_w$ ) following a regression analysis. For the sections failed below their yield  
 537 load, a separate expression was proposed while for the sections achieve a greater strength above  
 538 their yield load due to the pronounced strain hardening effect of stainless steel, another separate  
 539 expression was proposed with an upper limit based on the FE results. Table 15 summarises the  
 540 expressions proposed herein for the web shear buckling coefficient ( $\chi_w$ ) while all experimental  
 541 and FE data points are compared with the proposed EN1993-1-4 [16] curve for the web shear  
 542 buckling coefficient ( $\chi_w$ ) in Figure 18.

543

544 Table 15: Proposed web shear buckling reduction factor,  $\chi_w$  for webs with rigid end post for  
 545 EN1993-1-4 [16].

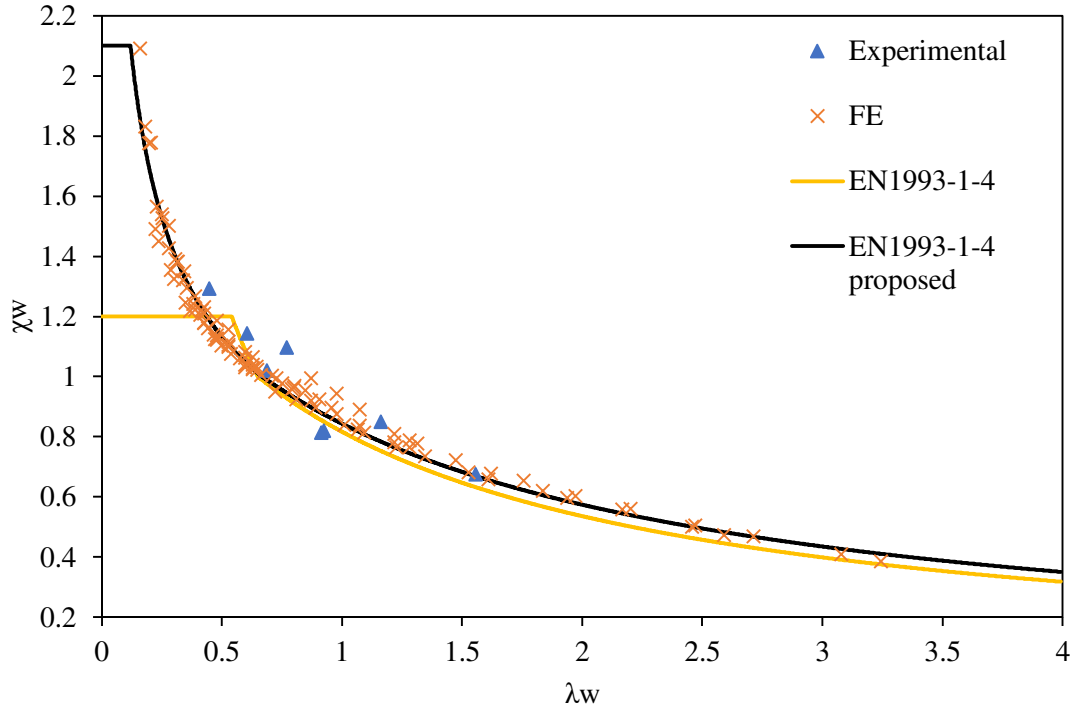
	$\chi_w$
$\bar{\lambda}_w \leq 0.12$	2.1
$0.12 < \bar{\lambda}_w < 0.667$	$0.839/\bar{\lambda}_w^{0.433}$
$\bar{\lambda}_w \geq 0.667$	$1.797/(1.13 + \bar{\lambda}_w)$

546

547 From Figure 18, it can be seen that proposed expressions for the web shear buckling coefficient  
 548 ( $\chi_w$ ) in EN1993-1-4 [16] considering stainless steel LCBs, were able to capture the shear  
 549 capacity well throughout the web slenderness ( $\bar{\lambda}_w$ ) range. From Table 16, it is highlighted that  
 550 compared to the current EN1993-1-4 [16] shear design provisions, proposed expressions  
 551 enhance the prediction accuracy specially in the compact region ( $\bar{\lambda}_w < 0.667$ ) emphasising the  
 552 ability to capture the inelastic reserve capacity available in stainless steel LCBs in shear.

553

554



555 Figure 18: Comparison of experimental and FE shear capacities with proposed curve for web  
 556 shear buckling coefficient ( $\chi_w$ ).

557

558 Table 16: Comparison of FE and experimental shear capacities with EN1993-1-4 [16]  
 559 predictions.

	$V_{FE \text{ (\& Exp.)}} / V_{EN1993-1-4}$	$V_{FE \text{ (\& Exp.)}} / V_{EN1993-1-4}$ <i>Proposed</i>
$\bar{\lambda}_w < 0.667$		
Mean	1.07	1.00
COV	0.157	0.039
$\bar{\lambda}_w \geq 0.667$		
Mean	1.07	1.02
COV	0.040	0.041
Overall		
Mean	1.07	1.01
COV	0.118	0.042

560

561 7.2 Proposed DSM shear design rules

562 In this section modifications made to the current DSM shear design provisions to enhance the  
 563 shear capacity prediction accuracy of stainless steel LCBs are detailed. Firstly, using Eqs. (14)-  
 564 (23), applicability of the current provisions were assessed and then Eq. (15) was recalibrated  
 565 and fitted to the experimental and FE data points by following a regression analysis.  
 566 Furthermore, another equation was proposed to capture the inelastic reserve capacity of the  
 567 compact sections with an upper limit. Therefore, this study suggests Eqs. (24)-(26) to be  
 568 employed instead of Eqs. (14) and (15) in the DSM shear design provisions for stainless steel  
 569 LCBs.

570 
$$\frac{V_b}{V_y} = 2 \text{ for } \lambda \leq 0.122 \quad (24)$$

571 
$$\frac{V_b}{V_y} = \frac{0.795}{\lambda^{0.439}} \text{ for } 0.122 < \lambda \leq 0.592 \quad (25)$$

572 
$$\frac{V_b}{V_y} = \left[ 1 - 0.213 \left( \frac{1}{\lambda^2} \right)^{0.35} \right] \left( \frac{1}{\lambda^2} \right)^{0.35} \text{ for } \lambda > 0.592 \quad (26)$$

573 where all the notations are defined in the Section 6.2.

574

575 Table 17: Comparison of FE and experimental shear capacities with DSM predictions.

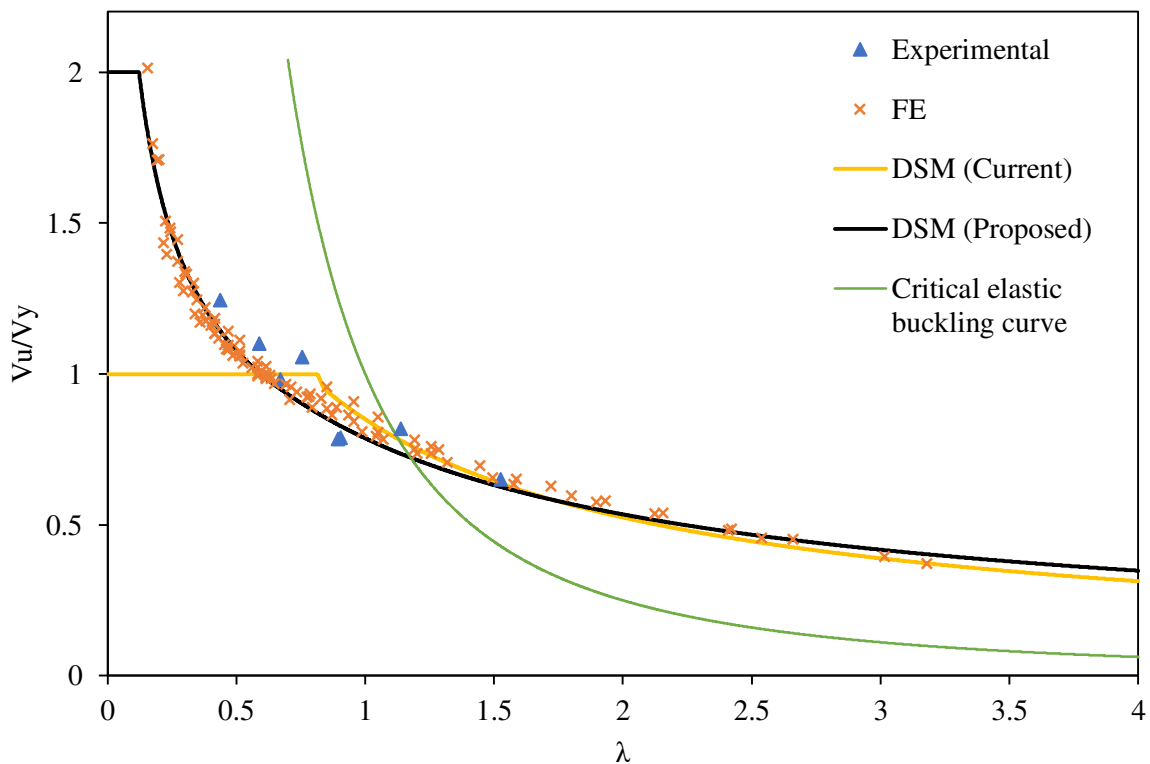
	$V_{FE \text{ (\& Exp.)}} / V_{DSM}$	$V_{FE \text{ (\& Exp.)}} / V_{DSM}$ <i>Proposed</i>
$\bar{\lambda}_w \leq 0.592$		
Mean	1.24	1.00
COV	0.175	0.040
$\bar{\lambda}_w > 0.592$		
Mean	1.00	1.04
COV	0.054	0.043
Overall		
Mean	1.11	1.02
COV	0.176	0.047

576

577

578 Current and proposed DSM shear capacity predictions are also compared in Table 17. From  
 579 Table 17, it can be seen that the newly proposed shear design equations significantly enhance  
 580 the prediction accuracy over the existing shear design equations. Also, unlike the existing  
 581 provisions, the new provisions are able to predict the inelastic reserve capacity of compact  
 582 sections. Figure 19 illustrates the comparison of experimental and FE shear capacities for  
 583 stainless steel LCBs against the newly proposed DSM design curve. In addition, the critical  
 584 elastic shear buckling curve is included in Figure 19 to demonstrate the available post-buckling  
 585 strength in slender sections.

586



587 Figure 19: Comparison of experimental and FE shear capacities with proposed DSM shear  
 588 design curve.

589

### 590 7.3 Reliability analysis

591 In order to assess the applicability of proposed EN1993-1-4 [16] and DSM shear design rules  
 592 a reliability analysis was carried out by following the method suggested in North American  
 593 specification for cold-formed steel structures [41]. In this method the capacity reduction factor

594 ( $\phi_v$ ) is calculated considering the effect of material and geometric variations as given by Eq.  
 595 (27).

$$596 \quad \phi_v = 1.52 M_m F_m P_m e^{-\beta_0 \sqrt{(V_m^2 + V_f^2 + C_p V_p^2 + V_q^2)}} \quad (27)$$

597 where  $M_m$  and  $V_m$  are the mean and COV of the material factor, respectively and taken as 1.1  
 598 and 0.1, respectively.  $F_m$  and  $V_f$  are the mean and COV of the fabrication factor, respectively  
 599 and taken as 1.0 and 0.05, respectively.  $P_m$  and  $V_p$  (not less than 0.065) are the mean and COV  
 600 of the experimental or FE to predicted ratio, respectively.  $V_q$  is the COV of the load effect  
 601 taken as 0.21.  $\beta_0$  is the target reliability index taken as 2.5.  $C_p$  is the correction factor and is  
 602 calculated using Eq. (28).

$$603 \quad C_p = \left[ 1 + \frac{1}{n} \right] \left[ \frac{m}{m-2} \right] \quad (28)$$

604 where  $n$  is the number of data points and  $m$  is the number of degrees of freedom, taken as  $n-1$ .

605 Considering all the experiments and FE models for stainless steel LCBs in shear reliability of  
 606 the proposed design rules were assessed. For the proposed EN1993-1-4 [16] design rules taking  
 607  $P_m=1.01$  (from Table 16) and  $V_p=0.065$  (recommended minimum value) resulted in  $\phi_v=0.91$ .  
 608 And for the developed DSM shear design rules adopting  $P_m=1.02$  (from Table 17) and  
 609  $V_p=0.065$  (recommended minimum value) resulted in  $\phi_v=0.92$ . Therefore, for both proposals  
 610 a capacity reduction factor ( $\phi_v$ ) of 0.90 is recommended.

## 611 **8 Concluding remarks**

612 This paper presents the details of testing and numerical modelling of stainless steel LCBs in  
 613 shear. Developed FE models were validated using the test results and highlighted the capability  
 614 of FE models to predict shear capacities, elastic shear buckling loads, and failure modes with  
 615 a reasonably good accuracy. From the FE results, it is also highlighted that there is significant  
 616 post-buckling strength in slender stainless steel LCBs in shear. The FE models of cold-formed  
 617 steel LCBs in shear were also developed and validated. Employing these cold-formed steel and  
 618 stainless steel FE models, inelastic reserve capacity envisaged in compact stainless steel LCBs  
 619 in shear was highlighted. It has been shown that when  $d_1/t$  ratio is less than 28, more than 40  
 620 % strength increment exists in compact sections due to the strain hardening effect of stainless  
 621 steel. 100 stainless steel FE models were developed using the validated FE models in order to  
 622 assess the applicability of current EN1993-1-4 [16] and DSM shear design rules for the

623 stainless steel LCBs. Current shear design rules were found to be too conservative specially for  
624 compact sections. Therefore, existing shear design rules were then modified to enhance the  
625 prediction accuracy. A set of expressions to predict web shear buckling coefficient ( $\chi_w$ ) in  
626 EN1993-1-4 [16] were proposed while detailing the modified and new equations for DSM  
627 shear design rules. It is worth to note that both proposed EN1993-1-4 [16] and DSM shear  
628 design rules are able to capture the available inelastic reserve capacity in compact stainless  
629 steel LCBs, unlike the existing shear design rules. However, more experimental data on the  
630 inelastic reserve capacity is recommended and is currently underway by the authors to enhance  
631 the understanding of this behaviour in compact sections.

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