Design of welded stainless steel I-shaped members subjected to shear

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Abstract: This paper aims to develop design methods for the shear strength of welded stainless steel I-shaped members. The current American AISC Design Guide 27 has been published for Structural Stainless Steel, and it refers to the specification ANSI/AISC 360-16 for structural steel buildings for the shear strength without considering tension field action (TFA). All the available test data on stainless steel plate girders were collected from literatures and employed to assess the provisions in the Design Guide 27, as well as the codified ones in Eurocode 3 Part 1.4. Based on the test results and obtained comparisons, two new proposals for unstiffened webs or webs with transverse stiffeners widely spaced (without TFA), and for interior webs with stiffeners spaced at $3h_w$ or smaller (considering TFA) were developed and presented to match the format of the expressions in ANSI/AISC 360-16 for stainless steel webs under shear. Reliability analysis was further performed to calculate the Load and Resistance Factor Design (LRFD) resistance factors determined by the new proposals and to further justify the target factor in AISC Design Guide 27 for structural stainless steel.

Keywords: Stainless steel; Shear design; I-shaped member; Tension field action; Reliability analysis

1. Introduction

Extensive experimental and analytical studies have been conducted on the shear behaviour of steel I-shaped members, and it is well recognised that the critical buckling and post-buckling strengths of web panels, and the frame action of flanges contribute substantially to the ultimate shear strength of I-shaped members. Various idealisations for characterising the post-buckling strength have been proposed by many scholars [1-7]. The post-buckling tension field theory was firstly established by Basler [1], where the tension field was assumed to develop throughout the whole web panel and the ultimate shear strength could be calculated by the sum of the critical and post-critical strength of the web panel. This method has been incorporated into the American specification ANSI/AISC 360-16 [8] for interior web panels with the aspect ratio less than or equal to 3.0 considering the tension field action (TFA). Another widely appreciated theoretical model is the rotated stress field method developed by Höglund [3], which is based on supposing that the principal tensile stress increases subsequent to critical buckling inducing the rotation of the principal stress, and the ultimate strength is derived by applying the von Mises yield criterion. The simplified formulae for the web shear post-buckling strength recommended by Höglund [3] formed the basis of the design provisions in ANSI/AISC 360-16 [8] for webs without TFA. Meanwhile, the Eurocode 3 Part 1.5 (EN 1993-1-5) [9] for plated structural elements employs the rotated stress method, yet takes into account the contributions from both the web panel and flanges.

These conventional design rules for carbon steel I-shaped members were derived by assuming elastic, perfectly plastic material behaviour, which can lead to inaccurate predictions for stainless steel I-shaped members due to the nonlinear and strain hardening characteristics of the material [10]. A great number of research studies on structural stainless steel have been motivated during the past two decades to develop alternative design methods [11-16]. However, the Eurocode 3 Part 1.4 (EN 1993-1-4+A1) [17] is the only available standard in English providing specific rules for shear design of welded stainless steel I-shaped members. The shear design formulae in EN 1993-1-4+A1 [17] were proposed following the provisions in EN 1993-1-5 [9], and have been validated by several experimental tests on stainless steel plate girders [18-23]. Though the American specifications SEI/ASCE 8-02 [24] and the Australian and New Zealand standard AS/NZS 4673:2001 [25] have been published for structural stainless steel, both of them can only apply to the design of cold-formed members, which may differ significantly from welded members. More recently, the AISC Design Guide 27 [26] for structural stainless steel covering hot-rolled and welded sections was released, which refers to the framework of ANSI/AISC 360-16 [8] for carbon steels and provides necessary modifications to suit the experimental data of stainless steel members. However, no separate design formulae have been recommended for the shear design of welded I-shaped members in the AISC Design Guide 27 [26], in which the existing ANSI/AISC 360-16 [8] formulae without considering TFA are provided.

Therefore, the primary intention of this paper is to collect available test data from published literatures and thereby to perform experimental assessment of the ANSI/AISC 360-16 [8] provisions in predicting the shear strength for welded stainless steel I-shaped members. Two design proposals for welded stainless steel I-shaped members were newly derived and presented in the format of the related expressions in ANSI/AISC 360-16 [8], which are expected to be included in future revisions of the AISC Design Guide 27. Further reliability analysis was carried out to acquire the Load and Resistance Factor Design (LRFD) resistance factors according to the procedure provided in AISC Design Guide 27 [26] for structural stainless steel.

2. Existing tests on stainless steel I-shaped members under shear

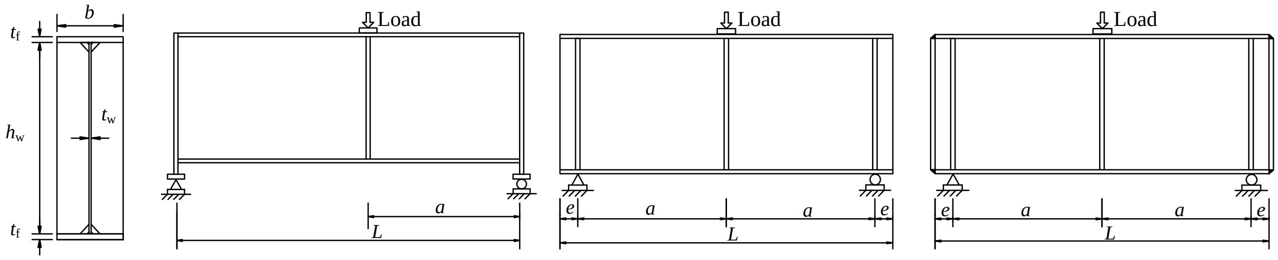
Experimental tests on welded stainless steel I-shaped members subjected to shear force have been carried out by a number of researchers. Specifically, Olsson [18] presented an experimental programme of eight austenitic and duplex stainless steel plate girders, and proposed a revised design approach based on the rotated stress field method, which was incorporated into the previous version of EN 1993-1-4 [27] for determining the shear strength of stainless steel plate girders by revising the provisions in the pre-standard ENV 1993-1-4 [28]. Real et al. [19] conducted nine tests on austenitic stainless steel plate girders and developed calculation formulae for predicting the critical shear buckling stress of stainless steel webs, taking into account the influence of material nonlinearity. Meanwhile, Estrada et al. [20] reported experimental results on eight austenitic stainless steel plate girder specimens and established new design methods for prediction of the ultimate shear strength [21] with either rigid or non-rigid end posts. More recently, Saliba and Gardner [22] performed nine tests on lean duplex stainless steel plate girders with rigid end posts, and proposed modified expressions to predict the ultimate shear strength of stainless steel plate girders [29], which have been incorporated into the current EN 1993-1-4+A1 [17]. Chen et al. [23] carried out an experimental study on a total of six austenitic and duplex stainless steel plate girders, and proposed a design treatment on predicting the ultimate shear strength that accounted for the effective restraints from the flanges and the rigid/non-rigid end posts.

The aforementioned experimental tests were collected and are summarised in Table 1 (including a total of forty tests), where F_y is nominal material yield strength of web panels, $V_{u,Test}$ and $V_{cr,Test}$ are the reported ultimate shear strength and critical shear buckling strength, respectively, and other geometric symbols are defined with reference to Fig. 1. These tested specimens made of three types of stainless steel alloys – austenitic, duplex and lean duplex grades, were designed with the web aspect ratio α (defined as a/h_w) varying between 1.00 to 3.25. Meanwhile, all tested plate girders had two web panels stiffened by different end post configurations, as presented in Fig. 1, twenty-five of which included double-sided bearing stiffener at end supports only, which were taken as non-rigid end post condition, while the other fifteen adopted rigid end posts consisting the double-sided bearing stiffener and an end cover plate. It is generally accepted that the rigid end posts are capable of providing sufficient flexural rigidity for the development of TFA in the web panels, hence the webs with rigid end posts are treated as interior web panels, while those with non-rigid end posts are considered as end web panels. Besides, it is worth noting that three tests from Saliba and Gardner [22] (marked as Case I in the last column of Table 1) exhibited bending dominant failure modes, and therefore have not been used in further assessment and analysis presented in this paper.

Table 1

Summarised experimental data of welded stainless steel I-shaped members under shear

Source	Material	I-shaped members	F_y (MPa)	L (mm)	a (mm)	h_w (mm)	b (mm)	t_w (mm)	t_f (mm)	a/h_w	$V_{u,Test}$ (kN)	$V_{cr,Test}$ (kN)	Case
Non-rigid end post													
Olsson [18]	1.4301	SB 4301:1	297.0	1049	449	146	200	4.00	11.90	3.08	178.5	-	
		SB 4301:2	297.0	2100	901	297	199	4.00	11.90	3.03	190.0	-	
		SB 4301:3	297.0	2998	1200	597	200	4.00	11.90	2.01	226.1	-	
		SB 4301:4	297.0	3997	1600	793	201	4.00	12.30	2.02	242.3	-	
	1.4462	SB 4462:1	573.0	1051	450	148	200	4.00	13.10	3.04	269.1	-	
		SB 4462:2	573.0	2100	900	298	200	4.00	13.10	3.02	294.5	-	
		SB 4462:3	573.0	2996	1200	597	203	4.00	13.00	2.01	366.3	-	
		SB 4462:4	573.0	3997	1600	795	202	4.00	13.00	2.01	388.0	-	
Real et al. [19]	1.4301	ad1w8	323.3	1160	500	500	200	8.00	20.00	1.00	804.0	-	
		ad1w6	323.4	1160	500	500	200	6.00	20.00	1.00	531.0	-	
		ad1w4	301.4	1160	500	500	200	4.00	20.00	1.00	353.0	-	
		ad15w8	323.3	1660	750	500	200	8.00	20.00	1.50	756.0	-	
		ad15w6	323.4	1660	750	500	200	6.00	20.00	1.50	484.0	-	
		ad15w4	301.4	1660	750	500	200	4.00	20.00	1.50	284.0	-	
		ad2w8	323.3	2160	1000	500	200	8.00	20.00	2.00	714.0	-	
		ad2w6	323.4	2160	1000	500	200	6.00	20.00	2.00	467.0	-	
		ad2w4	301.4	2160	1000	500	200	4.00	20.00	2.00	243.0	-	
Estrada et al. [20]	1.4301	nr700ad15	301.4	2360	1050	700	170	4.00	20.00	1.50	309.21	150	
		nr600ad2	301.4	2660	1200	600	170	4.00	20.00	2.00	260.65	180	
		nr500ad25	301.4	2760	1250	500	170	4.00	20.00	2.50	228.05	175	
		nr400ad325	301.4	2860	1300	400	170	4.00	20.00	3.25	217.9	175	
Chen et al. [23]	1.4301	V-304-300ad1	289.2	798.8	299.2	299.5	134	3.82	11.85	1.00	253.2	161	
		V-304-500ad1.5	279.7	1696.6	747.6	498.6	150.2	4.11	11.85	1.50	243.2	131	
	1.4462	V-2205-500ad1	539.6	1198.3	499	498.4	150.1	3.90	12.59	1.00	453.9	168	
		V-2205-500ad1.5	539.6	1698.3	748.7	499.4	150.1	3.90	12.59	1.50	385.9	127	
Rigid end post													
Estrada et al. [20]	1.4301	r700ad15	301.4	2360	1050	700	170	4.00	20.00	1.50	327.17	150	
		r600ad2	301.4	2660	1200	600	170	4.00	20.00	2.00	262.92	180	
		r500ad25	301.4	2760	1250	500	170	4.00	20.00	2.50	236.54	180	
		r400ad325	301.4	2860	1300	400	170	4.00	20.00	3.25	215.33	175	
Saliba and Gardner [22]	1.4162	I-600×200×12×4-1	486.0	1360	600	598.8	200.1	4.10	12.40	1.00	562.0	-	
		I-600×200×12×6-1	506.0	1360	600	599.9	199.8	6.20	12.30	1.00	888.0	-	
		I-600×200×12×8-1	431.0	1360	600	600.3	200.1	8.20	12.50	1.00	1326.0	-	
		I-600×200×12×10-1	433.0	1360	600	599.2	200.1	10.20	12.40	1.00	1838.0	-	I
		I-600×200×12×4-2	486.0	2560	1200	600	200.1	4.10	12.60	2.00	396.0	-	
		I-600×200×12×6-2	506.0	2560	1200	600.9	200	6.00	12.30	2.00	682.0	-	
		I-600×200×12×8-2	431.0	2560	1200	600	199.8	8.40	12.30	2.00	976.0	-	
		I-600×200×12×10-2	433.0	2560	1200	600.1	200.4	10.60	12.60	2.00	1162.0	-	I
		I-600×200×15×15-2	564.0	2560	1200	599	200.1	15.00	15.30	2.00	1801.0	-	I
Chen et al. [23]	1.4301	V-304-R500ad1	279.7	1198	499.4	498.6	150.1	4.11	11.85	1.00	322.2	153	
	1.4462	V-2205-R500ad1	539.6	1198.9	498.7	498.9	150.1	3.90	12.59	1.00	512.7	160	



(a) Cross-section (b) Non-rigid end post (Olsson) (c) Non-rigid end post (d) Rigid end post

Fig. 1. Geometry of the tested I-shaped members with different end stiffeners

3. Assessment of design methods for predicting the shear strength

3.1 General

The collected experimental data were used to evaluate the design methods for predicting the shear strength in AISC Design Guide 27 [26] and EN 1993-1-4+A1 [17] for structural stainless steel. The assessment is represented by the ratio of the test to predicted shear strength R ($R = V_{u,Test}/V_{u,pre}$), and a value of R higher than 1.0 indicates that the prediction is on the safe side. It should be noted that all predictions were calculated by referring to the codified provisions with all partial factors set to unity and using the experimentally measured material and geometric properties.

3.2 American AISC Design Guide 27

The Design Guide 27 [26] refers to the specification ANSI/AISC 360-16 [8] for the shear design of members. It should be noticed that, two different design methods for predicting the web shear post-buckling strength are presented in ANSI/AISC 360-16 [8]: the first of which is used for members with unstiffened webs, members with transverse stiffeners spaced wider than $3h_w$, and end panels of members with transverse stiffeners spaced closer than $3h_w$ (named without TFA), and the second one is for interior panels of members with stiffeners spaced at $3h_w$ or smaller (with TFA).

The first shear strength prediction method given in ANSI/AISC 360-16 [8] apply when post-buckling strength develops due to stress redistribution but classical TFA is not developed in web. It may be conservatively applied to any web panel but is mainly used for unstiffened webs, end web panels and webs with the aspect ratio exceeds 3.0 without considering TFA. The current codified expressions were proposed by Daley et al. [7] by adapting Höglund’s rotated stress field method [3] to plate girders with non-rigid end posts. The nominal shear strength V_{n1} is defined by Eq. (1), and the shear post-buckling strength factor C_{v1} is given by Eqs. (2) – (3).

$$V_{n1}=0.6F_y A_w C_{v1} \tag{1}$$

$$C_{v1}=1.0 \quad \text{if } h_w/t_w \le 1.10\sqrt{Ek_v/F_y} \tag{2}$$

$$C_{v1}=1.10\frac{\sqrt{Ek_v/F_y}}{h_w/t_w} \quad \text{if } h_w/t_w > 1.10\sqrt{Ek_v/F_y} \tag{3}$$

where the plate shear buckling coefficient k_v is determined by

$$\begin{aligned} k_v &= 5+\frac{5}{\left(a/h_w\right)^2} \\ &=5.34 \text{ when } a/h_w > 3.0 \end{aligned} \tag{4}$$

The comparison between the summarised test data and predicted strengths is presented in Table 2, and for illustration purposes, the obtained values of the ratio of the test to predicted shear strength R are plotted against the slenderness factor $\left(h_w/t_w\right)/\sqrt{Ek_v/F_y}$, as shown in Fig. 2. Clearly, it can be noted that the ANSI/AISC 360-16 method without

TFA ($R_{n1} = V_{u,Test}/V_{n1}$) is incapable of providing accurate and satisfactory shear strength predictions for stainless steel I-shaped members. Regarding the webs with lower slenderness factor (approximately less than 1.0), the shear strengths are underpredicted, which is mainly due to the absence of accounting for the strain hardening benefits of stainless steels. While for web panels with intermediate slenderness factor between 1.0 and 2.0, this method leads to overestimated shear strength predictions. The unsafe estimation is attributed to the negative impact of material nonlinearity of stainless steels, since the stress level of member with intermediate slenderness corresponds to the range between the proportional limit and the 0.2% proof stress (nominal yield strength), wherein the material nonlinearity is pronounced, yet it is not considered when using the idealised elastic, perfectly plastic material model for carbon steel. Moreover, for webs with higher values of slenderness factor, the ANSI/AISC 360-16 method without TFA yields relatively conservative strength predictions, especially in case of the rigid end post condition, since the strengthening effect from rigid end posts has not been explicitly considered. It is therefore highlighted the necessity to take into account the material nonlinearity and strain hardening of stainless steel alloys in the prediction of shear strength of I-shaped members.

For interior web panels with $a/h_w \leq 3.0$, the ANSI/AISC 360-16 provides the other method for the shear strength considering the TFA. This approach is based on the analytical model established by Basler [1], taking into account the critical shear buckling strength plus the tension field contribution to the shear strength. The nominal shear strength V_{n2} is then expressed by Eq. (5), in which the shear buckling coefficient C_{v2} is given by Eqs. (6) – (8).

$$V_{n2}=0.6F_yA_w\left[C_{v2}+\frac{1\text{-}C_{v2}}{1.15\sqrt{1+\left(a/h_w\right)^2}}\right] \tag{5}$$

$$C_{v2}=1.0 \quad \text{if } h_w/t_w \leq 1.10\sqrt{Ek_v/F_y} \tag{6}$$

$$C_{v2}=1.10\frac{\sqrt{Ek_v/F_y}}{h_w/t_w} \quad \text{if } 1.10\sqrt{Ek_v/F_y} < h_w/t_w < 1.37\sqrt{Ek_v/F_y} \tag{7}$$

$$C_{v1}=1.51\frac{Ek_v/F_y}{\left(h_w/t_w\right)^2} \quad \text{if } h_w/t_w \geq 1.37\sqrt{Ek_v/F_y} \tag{8}$$

It has to be noted that the current ANSI/AISC 360-16 provisions neglect the possible interaction effect between bending and shear strength by limiting the cross-sectional flange-to-web proportion [30], and the interior web panel is designed by considering the TFA. These limits are specified as $2A_w/(A_{ft}+A_{fc}) \leq 2.5$ or $h_w/b_{ft} \leq 6$ or $h_w/b_{fc} \leq 6$. Otherwise, a partial TFA method (as given by Eq. (9)) is recommended without the need to account for the bending and shear interaction.

$$V_{n2}=0.6F_yA_w\left[C_{v2}+\frac{1\text{-}C_{v2}}{1.15\left(a/h_w+\sqrt{1+\left(a/h_w\right)^2}\right)}\right] \tag{9}$$

The ultimate shear strengths of tested plate girders with rigid end posts and with aspect ratio less than 3.0 were utilised to compare with the predicted results. It is revealed from Table 2 and Fig. 2 that similar overpredicted results were obtained by ANSI/AISC 360-16 method with TFA for webs with intermediate slenderness factors. However, for the plate girders designed with higher values of slenderness factor, this method with TFA gave less conservative shear strengths for stainless steel plate girders with rigid end posts than the one without TFA. It is indicated that more accurate and economical design can be achieved by considering the TFA, yet improved revision is still needed for plate girders with intermediate slender web panels.

3.3 Eurocode EN 1993-1-4+A1

The current EN 1993-1-4+A1 [17] for structural stainless steel employs a uniform methodology for predicting the shear strength of web panels with and without rigid end posts based on the rotated stress field theory developed by Höglund [3] and adapted to stainless steels initially by Olsson [18] and further by Saliba et al. [29], following the provisions in EN 1993-1-5 [9] for carbon steel. The ultimate shear strength of web panels V_{bw} is given as

$$V_{bw} = \chi_w h_w t_w \frac{F_y}{\sqrt{3}} \tag{10}$$

wherein different expressions for calculating the shear buckling reduction factor χ_w are presented for the two different end post conditions. For plate girders with non-rigid end posts,

$$\chi_w = \eta \quad \text{if } \bar{\lambda}_w \le 0.65/\eta \tag{11}$$

$$\chi_w = 0.65/\bar{\lambda}_w \quad \text{if } 0.65/\eta < \bar{\lambda}_w < 0.65 \tag{12}$$

$$\chi_w = 1.19/\left(0.54 + \bar{\lambda}_w\right) \quad \text{if } \bar{\lambda}_w \ge 0.65 \tag{13}$$

For plate girders with rigid end posts, Eqs. (11) – (12) are also included with

$$\chi_w = 1.56/\left(0.91 + \bar{\lambda}_w\right) \quad \text{if } \bar{\lambda}_w \ge 0.65 \tag{14}$$

in which the web slenderness $\bar{\lambda}_w$ is defined according to Eqs. (15) – (17).

$$\bar{\lambda}_w = \frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}} \tag{15}$$

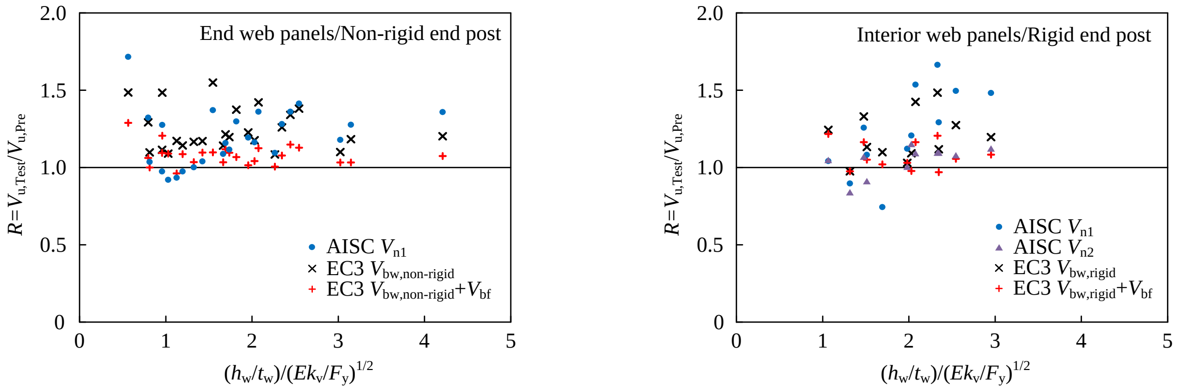
$$\varepsilon = \sqrt{\frac{235}{F_{yw}} \frac{E}{210000}} \tag{16}$$

$$\begin{aligned} k_\tau &= 5.34 + 4\left(h_w/a\right)^2 \quad \text{when } a/h_w \ge 1 \\ k_\tau &= 4 + 5.34\left(h_w/a\right)^2 \quad \text{when } a/h_w < 1 \end{aligned} \tag{17}$$

Furthermore, the shear contribution from the flanges is also taken into account in EN 1993-1-4+A1, if the bending capacity of flange M_f exceeds the applied bending moment M_{Ed}. The flange contribution is defined by Eq. (18) and the iteration process is required to obtain the optimum flange strength contribution.

$$V_{bf} = \frac{b_f t_f^2 F_{yf}}{c}\left(1 - \left(\frac{M_{Ed}}{M_f}\right)^2\right) \tag{18}$$

The evaluation of EN 1993-1-4+A1 for stainless steels is also illustrated in Table 2 and Fig. 2. Compared to the ANSI/AISC 360-16 provisions, the design method for shear strength of web panels in EN 1993-1-4+A1 provides more reasonable predictions while maintaining a simple design process. Moreover, closer agreement between the tests and predicted results has been achieved by accounting for strength contributions from both the web and flanges. Therefore, the provisions in EN 1993-1-4+A1 will be considered in the following section to develop new proposals for welded stainless steel I-shaped plate girders.



(a) End web panels/Non-rigid end post (b) Interior web panels/Rigid end post

Fig. 2. Comparison of test data with predictions from codified design methods

Table 2

Comparison of strength prediction ratios ($R = V_{u,Test}/V_{u,pre}$) for the codified design methods and new proposals

Source	I-shaped members	a/h_w	h_w/t_w	$V_{u,Test}$ (kN)	ANSI/AISC 360-16			EN 1993-1-4+A1			New proposals	
					$\frac{h_w/t_w}{\sqrt{Ek_v/F_y}}$	R_{n1}	R_{n2}	$\bar{\lambda}_w$	R_{web}	$R_{web+flange}$	$R_{n1,New}$	$R_{n2,New}$
Non-rigid end post												
Olsson (2001)	SB 4301:1	3.08	36.5	178.5	0.57	1.72	-	0.43	1.49	1.29	1.72	-
	SB 4301:2	3.03	74.3	190.0	1.20	0.97	-	0.92	1.14	1.09	1.10	-
	SB 4301:3	2.01	149.3	226.1	2.27	1.09	-	1.80	1.08	1.01	1.02	-
	SB 4301:4	2.02	198.3	242.3	3.03	1.18	-	2.40	1.10	1.03	1.03	-
	SB 4462:1	3.04	37.0	269.1	0.80	1.32	-	0.61	1.29	1.06	1.32	-
	SB 4462:2	3.02	74.5	294.5	1.67	1.09	-	1.28	1.14	1.03	1.10	-
	SB 4462:3	2.01	149.3	366.3	3.15	1.28	-	2.50	1.18	1.03	1.11	-
	SB 4462:4	2.01	198.8	388.0	4.21	1.36	-	3.34	1.20	1.07	1.12	-
Real et al. (2007)	ad1w8	1.00	62.5	804.0	0.81	1.04	-	0.67	1.10	1.00	1.04	-
	ad1w6	1.00	83.3	531.0	1.13	0.93	-	0.93	1.17	0.96	1.07	-
	ad1w4	1.00	125.0	353.0	1.55	1.37	-	1.28	1.55	1.10	1.41	-
	ad15w8	1.50	62.5	756.0	0.96	0.97	-	0.77	1.12	1.09	1.04	-
	ad15w6	1.50	83.3	484.0	1.33	1.00	-	1.07	1.17	1.04	1.09	-
	ad15w4	1.50	125.0	284.0	1.82	1.30	-	1.46	1.37	1.07	1.28	-
	ad2w8	2.00	62.5	714.0	1.03	0.92	-	0.82	1.09	1.09	1.03	-
	ad2w6	2.00	83.3	467.0	1.42	1.04	-	1.13	1.17	1.10	1.10	-
	ad2w4	2.00	125.0	243.0	1.95	1.19	-	1.55	1.23	1.01	1.15	-
Estrada et al. (2007)	nr700ad15	1.50	175.0	309.21	2.55	1.41	-	2.05	1.38	1.13	1.28	-
	nr600ad2	2.00	150.0	260.65	2.35	1.28	-	1.86	1.26	1.08	1.18	-
	nr500ad25	2.50	125.0	228.05	2.03	1.16	-	1.60	1.18	1.04	1.11	-
	nr400ad325	3.25	100.0	217.9	1.69	1.16	-	1.31	1.21	1.12	1.16	-
Chen et al. (2018)	V-304-300ad1	1.00	78.4	253.2	0.96	1.28	-	0.79	1.48	1.21	1.36	-
	V-304-500ad1.5	1.50	121.3	243.2	1.74	1.12	-	1.40	1.20	1.10	1.11	-
	V-2205-500ad1	1.00	127.8	453.9	2.08	1.36	-	1.72	1.42	1.12	1.29	-
	V-2205-500ad1.5	1.50	128.1	385.9	2.45	1.36	-	1.97	1.34	1.15	1.24	-
Rigid end post												
Estrada et al. (2008)	r700ad15	1.50	175.0	327.17	2.55	1.50	1.07	2.05	1.27	1.06	1.35	1.07
	r600ad2	2.00	150.0	262.92	2.35	1.29	1.09	1.86	1.12	0.97	1.19	1.10

	r500ad25	2.50	125.0	236.54	2.03	1.21	1.14	1.60	1.09	0.98	1.15	1.18
	r400ad325	3.25	100.0	215.33	1.69	1.14	-	1.31	1.10	1.02	1.15	-
	I-600×200×12×4-1	1.00	146.0	562.0	2.33	1.66	1.09	1.93	1.48	1.21	1.54	1.09
	I-600×200×12×6-1	1.00	96.8	888.0	1.51	1.08	0.91	1.25	1.13	1.05	1.12	0.97
	I-600×200×12×8-1	1.00	73.2	1326.0	1.07	1.04	1.04	0.88	1.24	1.22	1.19	1.16
Saliba and	I-600×200×12×10-1	1.00	58.7	1838.0	Bending dominant failure							
Gardner	I-600×200×12×4-2	2.00	146.3	396.0	2.95	1.48	1.12	2.34	1.20	1.08	1.30	1.12
(2013)	I-600×200×12×6-2	2.00	100.2	682.0	1.98	1.12	1.00	1.57	1.03	1.03	1.08	1.03
	I-600×200×12×8-2	2.00	71.4	976.0	1.32	0.90	0.83	1.04	0.98	0.98	0.97	1.00
	I-600×200×12×10-2	2.00	56.6	1162.0	Bending dominant failure							
	I-600×200×15×15-2	2.00	39.9	1801.0								
Chen et al.	V-304-R500ad1	1.00	121.3	322.2	1.48	1.26	1.06	1.22	1.33	1.16	1.32	1.15
(2018)	V-2205-R500ad1	1.00	127.9	512.7	2.08	1.54	1.09	1.72	1.42	1.16	1.46	1.10

4. New proposal for shear strength without TFA

By referring to the methodology used by Daley et al. [7] to develop the current ANSI/AISC 360-16 [8] method without TFA on the basis of Höglund's proposals [3] for steel plate girders with non-rigid end posts, new design formulae are derived herein in the format of ANSI/AISC 360-16 expressions for stainless steel plate girders. The new proposal for the shear strength without TFA is developed based on the equations in EN 1993-1-4+A1 for shear strength prediction of web panels V_{bw} with non-rigid end post. The relationship between the calculated factors employed in the European code and American specification is examined. The shear strength prediction for web panels in EN 1993-1-4+A1 (see Eq. (10)) is the product of the plastic shear strength $A_w F_y / \sqrt{3}$ and the shear buckling reduction factor χ_w, while in ANSI/AISC 360-16 (see Eq. (1)), the plastic shear strength is approximated as $0.6F_yA_w$, and the reduction factor C_{v1} is expressed as the function of $(h_w/t_w)/\sqrt{Ek_v/F_y}$, instead of the slenderness parameter $\bar{\lambda}_w$ in EN 1993-1-4+A1. In essence, the slenderness $\bar{\lambda}_w$ is the square root of the ratio of the shear yield strength τ_y to the elastic shear buckling stress $\tau_{cr,e}$. Thus, by approximating τ_y by $0.6F_y$ to match the ANSI/AISC 360-16 expression, the parameter $\bar{\lambda}_w$ could be rewritten as

$$\bar{\lambda}_w = \sqrt{\frac{\tau_y}{\tau_{cr,e}}} = \frac{h_w/t_w}{\sqrt{Ek_v/0.6F_y}} \frac{1}{\sqrt{\pi^2/12\left(1-\nu^2\right)}} \tag{19}$$

Substitution of Poisson's ratio ν equal to 0.3 in Eq. (19) leads to equivalent formulae for $C_{v1,EC3,Non\text{-}rig}$,

$$C_{v1,EC3,Non\text{-}rig} = \eta \quad \text{if } h_w/t_w \le 0.67\sqrt{Ek_v/F_y} \tag{20}$$

$$C_{v1,EC3,Non\text{-}rig} = \frac{0.80\sqrt{Ek_v/F_y}}{h_w/t_w} \quad \text{if } 0.67\sqrt{Ek_v/F_y} < h_w/t_w < 0.80\sqrt{Ek_v/F_y}h_w \tag{21}$$

$$C_{v1,EC3,Non\text{-}rig} = \frac{1.46\sqrt{Ek_v/F_y}}{0.66\sqrt{Ek_v/F_y} + h_w/t_w} \quad \text{if } h_w/t_w \ge 0.80\sqrt{Ek_v/F_y} \tag{22}$$

where η equal to 1.2 is recommended in EN 1993-1-4+A1 accounting for the strain hardening capacity of stainless steel alloys.

Based on the above formulae derived from EN 1993-1-4+A1, further modifications are suggested. For being consistent with ANSI/AISC 360-16 for structural steel at the maximum shear strength, the maximum value of C_{v1} is conservatively taken as 1.0, instead of 1.2 in EN 1993-1-4+A1. Therefore, Eqs. (20) and (21) could be merged as C_{v1}

= 1.0 in the yielding range ($h_w/t_w < 0.8\sqrt{Ek_v/F_y}$). Meanwhile, in view of the fact that the shear contribution from flanges is not explicitly considered, and the rigid and non-rigid end post conditions are not separately treated in the ANSI/AISC 360-16 specification without TFA, the coefficient C_{v1} in Eq. (22) is increased by approximately 5% in order to adapt to the experimental data. Hence the newly proposed expressions for the shear post-buckling strength factor $C_{v1,New}$ are given by Eqs. (23) – (24), where the plate shear buckling coefficient k_v defined by Eq. (4) is adopted as the same with the current ANSI/AISC 360-16 specification.

$$C_{v1,New} = 1.0 \quad \text{if } h_w/t_w \le 0.85\sqrt{Ek_v/F_y} \tag{23}$$

$$C_{v1,New} = \frac{1.55\sqrt{Ek_v/F_y}}{0.7\sqrt{Ek_v/F_y} + h_w/t_w} \quad \text{if } h_w/t_w > 0.85\sqrt{Ek_v/F_y} \tag{24}$$

The results summarised from the existing tests on stainless steel plate girders were employed to assess the new proposal for $C_{v1,New}$, as presented in Fig. 3, in which the nondimensional shear strength $V_{u,Test}/(0.6F_yA_w)$ and the predicted ratio $R = V_{u,Test}/V_{u,pre}$ are plotted against the slenderness factor $(h_w/t_w)/\sqrt{Ek_v/F_y}$. The predicted strengths are also listed in Table 2. Compared with the current expressions of C_{v1} in ANSI/AISC 360-16, the newly proposed method yields safe and more accurate shear strengths for web panels with intermediate values of slenderness, and provides less conservative predictions in case of web panels with high values of slenderness. It is therefore demonstrated that the proposed method without TFA can be adopted as an alternative approach for predicting the shear strength of welded stainless steel I-shaped members. Further statistical analysis will be performed in Section 6 to verify the reliability of the proposed expressions.

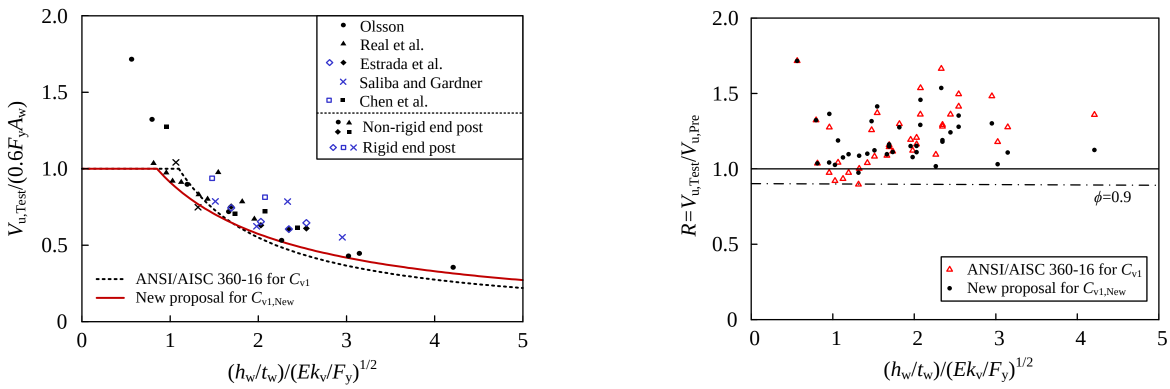


Fig. 3. Comparison of test results with codified and proposed methods without TFA

5. New proposal for shear strength with TFA

5.1 General

The comparison in Section 3 has demonstrated that for plate girders designed with slender interior web panels, the method considering TFA can provide superior shear strength predictions than the other one without TFA, new design formulae with TFA are to be developed in this section for stainless steel interior webs within the framework of Basler's tension field model. The original expression proposed by Basler [1] is given as

$$V_{n,Basler} = V_p\left[\tau_{cr,r}/\tau_y + \frac{\sqrt{3}}{2}\frac{1-\tau_{cr,r}/\tau_y}{\sqrt{1+(a/h_w)^2}}\right] \tag{25}$$

It can be noted that the shear buckling coefficient C_{v2} (see Eq. (5)) used for the ANSI/AISC 360-16 method with TFA is essentially similar to the ratio of the critical shear buckling stress $\tau_{cr,r}$ to the yield shear strength τ_y. Hence, the key point of developing a new method for stainless steel web panels is to establish expressions for the critical shear buckling strength considering the effect of the material nonlinearity. Moreover, among the existing studies on stainless steel plate girders, Chen et al. [23] presented experimentally determined critical shear buckling stresses based on the strain reversal method; Estrada et al. [20] obtained critical shear buckling stresses of the tested specimens by nonlinear numerical analysis, and Real et al. [19] conducted nonlinear numerical modelling of simply supported plates. These test/FE results are utilised to evaluate the theoretical and proposed methods for determining the critical shear buckling strength.

5.2 Critical shear buckling strength

The elastic solution to the shear buckling stress for a rectangular plate is defined by

$$\tau_{\mathrm{cr,e}}=k_{\mathrm{v}}\frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t_{\mathrm{w}}}{h_{\mathrm{w}}}\right)^2 \tag{26}$$

In view of the material nonlinearity of stainless steel, the critical shear buckling stress of a stainless steel plate can be obtained by introducing a plasticity reduction factor η.

$$\tau_{\mathrm{cr,r}}=\eta\tau_{\mathrm{cr,e}} \tag{27}$$

Various analytical approaches for calculating the plastic reduction factor η have been developed and served as the basis of codified provisions. The American specification SEI/ASCE 8-02 [18] for cold-formed stainless steel beam webs under shear employs the reduction factor $\eta = G_s/G_0$, which involves an iterative design procedure. The ENV 1993-1-4 [28] provides a calculation method for the critical shear buckling strength of stainless steel web panels, which was found to be derived using $\eta = G_t/G_0$ [22]. The shear stress-strain relationship for determining the secant shear modulus G_s and the tangent shear modulus G_t was developed by Carvalho et al. [31], which was derived from the typical stress-strain curve by introducing an affinity factor $\beta = 1.3$.

$$\gamma=\frac{\tau}{G_0}+0.002\beta\left(\frac{\tau}{\tau_{\mathrm{y}}}\right)^n \tag{28}$$

where n is the strain hardening exponent and G_0 is the initial shear modulus taken as $G_0 = E_0/2(1+\nu)$. The two shear moduli G_s and G_t can thus be calculated by

$$G_{\mathrm{s}}=\frac{\tau}{\gamma}=\frac{G_0}{1+0.002\beta\left(\tau^{\mathrm{n-1}}/\tau_{\mathrm{y}}^{\mathrm{n}}\right)G_0} \tag{29}$$

$$G_{\mathrm{t}}=\frac{d\tau}{d\gamma}=\frac{G_0}{1+0.002\beta n\left(\tau^{\mathrm{n-1}}/\tau_{\mathrm{y}}^{\mathrm{n}}\right)G_0} \tag{30}$$

Real et al. [19] conducted experimental and numerical studies which mainly focused on the analysis of the critical shear buckling stress of stainless steel webs, and they concluded that using $\eta = (G_t/G_0)^{1/2}$ could lead to closer agreement between the numerical and analytical results. Therefore, explicit solutions are to be proposed for predicting the critical shear buckling strength of stainless steel webs fitting this approach. The typical stress-strain diagram for stainless steel are divided into four different zones by referring to the corresponding levels of nonlinearity, as shown in Fig. 4: Zone 1 is defined if the yield strength can be attained in the web panel before the occurrence of initial shear buckling; Zone 2 lies between the proportional limit and yield stress, where the material nonlinearity leads to obvious stress reduction; in Zone 3 the initial shear buckling stress is somewhat influenced by the material nonlinearity but is still lower than the proportional limit, and in Zone 4 the rather low initial shear buckling stress is not influenced by the material nonlinearity.

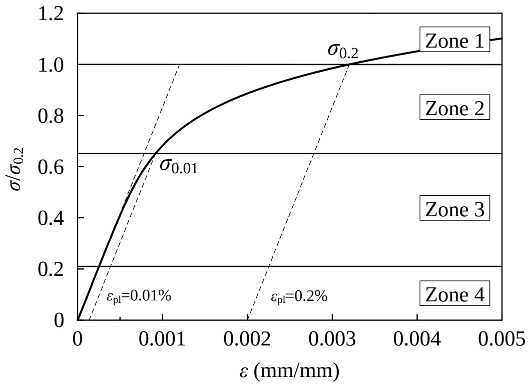
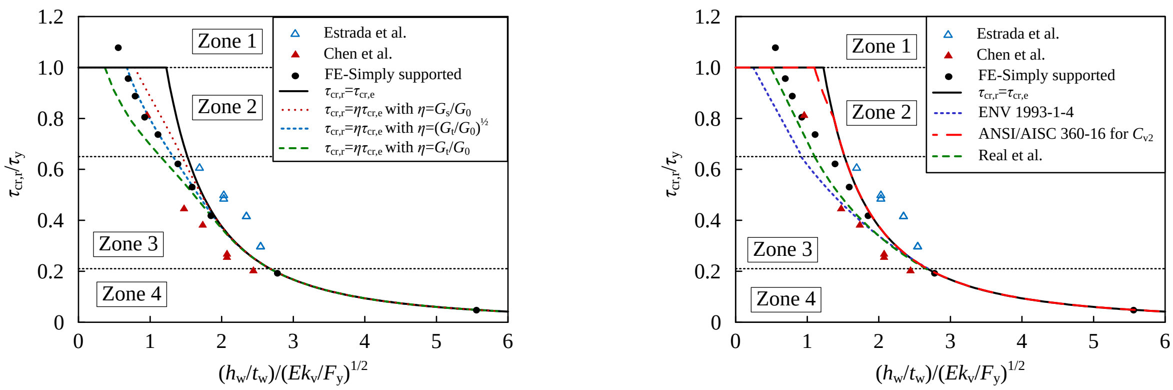


Fig. 4. Stress zones defined by Real et al. [19]

The aforementioned test/FE results on the critical shear buckling strength are utilised herein to evaluate the theoretical and proposed methods, as illustrated in Fig. 5, where the comparison is presented together with the four zones of different buckling stress levels proposed by Real et al. [19]. Among the three analytical methods, using $\eta = (G_t/G_0)^{1/2}$ results the best fit to the FE results from Real et al. [19] and the test results from Chen et al. [23] in Zone 2 ($0.65 < \tau_{cr,r}/\tau_y < 1.0$), where the initial shear buckling stresses are heavily influenced by the material nonlinearity. The data points in Zone 3 show considerable scatter, and this is mainly due to the different methods used to determine the critical buckling stress. Similar superior tendency can be achieved from the proposal by Real et al. [19], as shown in Fig. 5 (b), while the design curve in ENV 1993-1-4 provides rather conservative strength predictions. Furthermore, since the material nonlinearity is not explicitly considered in ANSI/AISC 360-16 for C_{v2}, it leads to overpredicted critical shear buckling strength values for stainless steel web panels. For webs with relatively higher slenderness, the predicted critical buckling stresses converge to the elastic buckling curve, which is independent of material nonlinearity.



(a) Theoretical methods

(b) Codified and proposed methods

Fig. 5. Comparison of test/FE results of critical shear buckling stress with existing methods

5.3 New proposal for critical shear buckling strength

The advantage of introducing $\eta = (G_t/G_0)^{1/2}$ into calculation of the critical shear buckling strength of stainless steel web panels has been highlighted in the previous sub-section, which is therefore utilised to develop further design approach for the shear strength. The use of the plasticity reduction factor η involves some lengthy iterations to acquire the shear buckling strength, therefore explicit calculation formulae are considered preferable for hand computation. Moreover, it is worth noting that the value of $\eta = (G_t/G_0)^{1/2}$ varies with the shear stress and depends on the strain hardening exponent n, which is a measure of the nonlinearity of the stress-strain curve (i.e., smaller value of n indicates a greater degree of nonlinearity). In the fourth edition of European Design manual for stainless steel [32] that is expected to be included in future revisions of Eurocode 3 Part 1.4, it is recommended that the exponent n should be taken as 14 for ferritic grades, 7 for austenitic grades and 8 for duplex grades, respectively. Therefore, the lowest value $n = 7$ is conservatively adopted for development of new proposals for both the critical shear buckling strength and the ultimate shear strength.

The newly proposed expressions are also divided into four zones, as shown in Fig. 6, according to different buckling stress levels. Compared to the zones defined by Real et al. [19], one modification has been made for the limit between Zone 2 and Zone 3. The proportional limit is taken as the 0.05% proof stress $\sigma_{0.05}$, instead of the previously used 0.01% proof stress $\sigma_{0.01}$, because it has been verified that a better representation of the exponent n, was achieved by using $\sigma_{0.05}$ [11]. Therefore, using the calculation of n in the European Design Manual for stainless steel: $n = \ln(4)/\ln(\sigma_{0.2}/\sigma_{0.05})$, the low bound of Zone 2 can be computed as $\sigma_{0.05}/\sigma_{0.2} = (1/4)^{1/n} \approx 0.82$, which is conservatively taken as 0.80 in the new proposal.

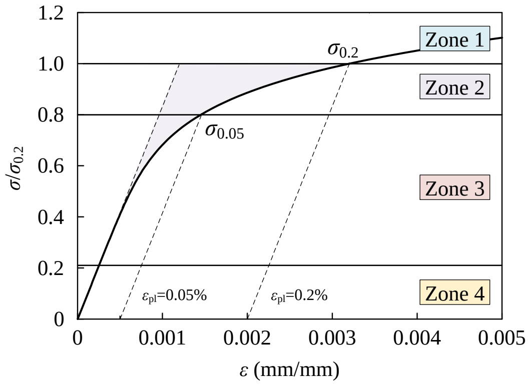
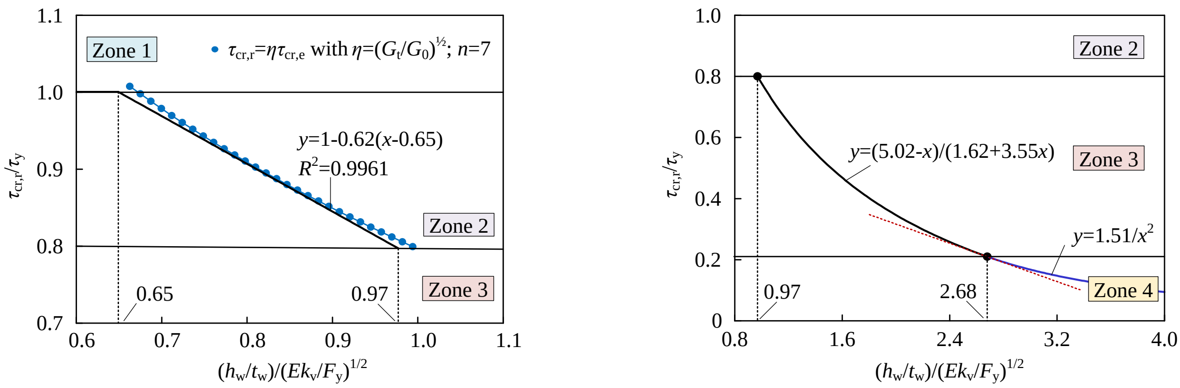


Fig. 6. Newly proposed stress zones



(a) Expression in Zone 2

(b) Expression in Zone 3

Fig. 7. Development of new proposal for critical shear buckling strength

The data points of $\tau_{cr,r}$ with $\eta = (G_t/G_0)^{1/2}$ obtained by iterations are plotted in Fig. 7. It can be seen that the relationship between the factor $(h_w/t_w)/\sqrt{Ek_v/F_y}$ and $\tau_{cr,r}/\tau_y$ can be represented by a simple linear model in Zone 2. Fig. 7 (a) shows the linear curve obtained by fitting the data points, and this straight line intersects the two limit lines ($\tau_{cr,r}/\tau_y$ = 1.0 and 0.8) at 0.65 and 0.97, respectively. In Zone 4 ($\tau_{cr,r}/\tau_y < 0.21$), where the initial shear buckling stress is rather low and is not effected by the material nonlinearity, the expression is taken the same as the elastic one. Moreover, the function in Zone 3 maintains the format of the expression in ENV 1993-1-4 [30]. It was derived by fitting the points in upper and lower limits plus keeping the same slope at the lower limit with the formula in Zone 4, in order to achieve continuity of the function, as presented in Fig. 7 (b). Consequently, the newly proposed expressions for determining the shear buckling coefficient $C_{v2,New}$ of stainless steel interior webs can be obtained as follows:

$$C_{v2,New} = 1.0 \quad \text{if } h_w/t_w \le 0.65\sqrt{Ek_v/F_y} \tag{31}$$

$$C_{\mathrm{v2,New}} = 1 - 0.62\left(\frac{h_{\mathrm{w}}/t_{\mathrm{w}}}{\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}}} - 0.65\right) \quad \text{if } 0.65\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}} < h_{\mathrm{w}}/t_{\mathrm{w}} \le 0.97\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}} \tag{32}$$

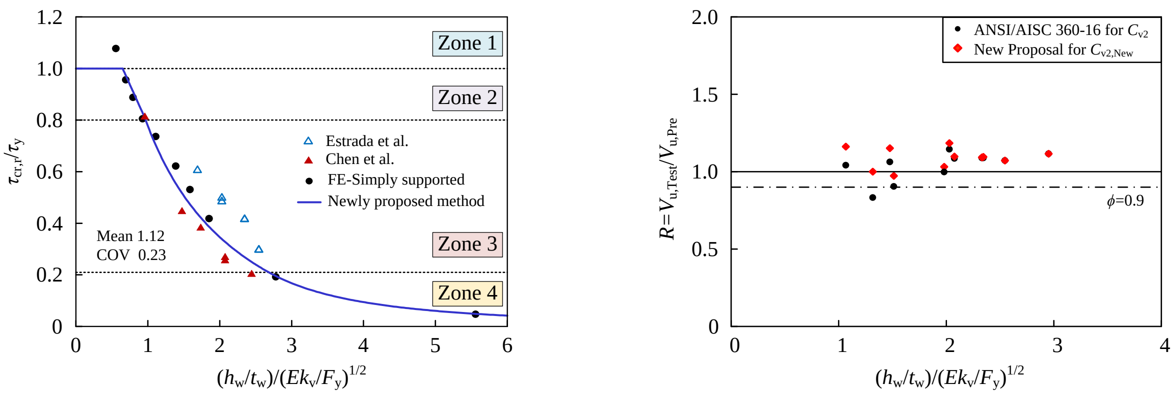
$$C_{\mathrm{v2,New}} = \frac{5.02\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}} - h_{\mathrm{w}}/t_{\mathrm{w}}}{1.62\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}} + 3.55 h_{\mathrm{w}}/t_{\mathrm{w}}} \quad \text{if } 0.97\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}} < h_{\mathrm{w}}/t_{\mathrm{w}} \le 2.68\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}} \tag{33}$$

$$C_{\mathrm{v2,New}} = 1.51\frac{Ek_{\mathrm{v}}/F_{\mathrm{y}}}{\left(h_{\mathrm{w}}/t_{\mathrm{w}}\right)^2} \quad \text{if } h_{\mathrm{w}}/t_{\mathrm{w}} > 2.68\sqrt{Ek_{\mathrm{v}}/F_{\mathrm{y}}} \tag{34}$$

The newly proposed expressions for $C_{\mathrm{v2,New}}$ are plotted in Fig. 8 (a) with the test/FE data points of the critical shear buckling stress, where the average value of test/FE to predicted ratios is equal to 1.12 with a corresponding coefficient of variation (COV) of 0.23. Despite the inherent scatter from experimentally determined data points, it can be concluded that satisfactory predictions of the critical shear buckling stress have been achieved for the stainless steel I-shaped members under shear.

5.4 Assessment of new proposal for shear strength with TFA

The newly proposed expressions for $C_{\mathrm{v2,New}}$ are substituted into Eq. (5) for calculating the ultimate shear strength of interior web panels considering the TFA. The accuracy of the new proposal is further assessed by means of the summarised test results for the ultimate shear strength, as illustrated in Fig. 8 (b) and listed in Table 2. Compared with the current ANSI/AISC 360-16 provisions for C_{v2}, the new proposal yields improved accuracy in predicting shear strengths for stainless steel web panels with intermediate slenderness, and maintains the consistency in case of web panels with higher slenderness. The reliability of the new proposal will be addressed in the following section.



(a) Critical shear buckling stress (b) Ultimate shear strength with TFA

Fig. 8. Comparison of test/FE results with codified and proposed method with TFA

6. Resistance factors

Reliability analysis was conducted on the newly proposed methods and the design methods codified in ANSI/AISC 360-16 for predicting the ultimate shear strength of stainless steel I-shaped members. The LRFD resistance factors of each method were calculated in accordance with the procedure given in Appendix B of the AISC Design Guide 27 [26]. The statistical uncertainties and the obtained resistance factors with a target reliability index β = 2.6 and a 'dead to live' load ratio of 1:3, are summarised in Table 3 and Table 4 for the methods without TFA and considering TFA, respectively. The statistical uncertainties are defined as follow: M_{m}, F_{m} and P_{m} are the mean values of the random variables in material, geometry and design assumptions, respectively, and V_{m} ,V_{f} and V_{p} are the corresponding COVs. $R_{\mathrm{m}}/R_{\mathrm{n}}$ is the product of these three mean values, while V_{R} is taken as the square-root-sum-of squares of these three

COVs. In general, the resistance factors determined by the new proposal are higher than those obtained by the current ANSI/AISC 360-16 method for carbon steel. Regarding the new proposal without TFA, the analysis indicates P_m = 1.184 and V_p = 0.136 with an average resistance factor $\phi_{calculated}$ = 1.163 for austenitic I-shaped members, while for duplex stainless steel members, involving both duplex and lean duplex grades, values of P_m = 1.218, V_p = 0.131, and $\phi_{calculated}$ = 1.019 are determined. Meanwhile, by adopting the proposed method with TFA, values of P_m = 1.216 and V_p = 0.039 for austenitic stainless steel are obtained, which leads to a resistance factor of 1.212. For duplex stainless steel, P_m = 1.009, V_p = 1.068 and $\phi_{calculated}$ = 0.962 are acquired. Moreover, it can be seen that the resulting resistance factors $\phi_{calculated}$ for both austenitic and duplex stainless steels exceed the recommended value of 0.90 in AISC Design Guide 27, demonstrating the applicability of the newly proposed design methods to the shear strength of welded stainless steel I-shaped members.

Table 3

Summary of reliability analysis results for design methods for web panels without TFA

Material	No. Test	M_m	F_m	P_m	R_m/R_n	V_m	V_f	V_p	V_R	$\phi_{calculated}$	Design methods without TFA
Austenitic	24	1.3	1.00	1.189	1.545	0.105	0.05	0.159	0.197	1.129	ANSI/AISC 360-16 for C_{v1}
		1.3	1.00	1.184	1.540	0.105	0.05	0.136	0.179	1.163	New Proposal for C_{v1}
Duplex /Lean duplex	13	1.1	1.00	1.276	1.404	0.105	0.05	0.172	0.208	1.005	ANSI/AISC 360-16 for C_{v1}
		1.1	1.00	1.218	1.340	0.105	0.05	0.131	0.175	1.019	New Proposal for C_{v1}

Table 4

Summary of reliability analysis results for design methods for interior web panels with $a/h_w \leq 3$ considering TFA

Material	No. Test	M_m	F_m	P_m	R_m/R_n	V_m	V_f	V_p	V_R	$\phi_{calculated}$	Design methods considering TFA
Austenitic	4	1.3	1.00	1.092	1.419	0.105	0.05	0.029	0.120	1.180	ANSI/AISC 360-16 for C_{v2}
		1.3	1.00	1.126	1.463	0.105	0.05	0.039	0.123	1.212	New Proposal for C_{v2}
Duplex /Lean duplex	7	1.1	1.00	1.009	1.110	0.105	0.05	0.097	0.151	0.880	ANSI/AISC 360-16 for C_{v2}
		1.1	1.00	1.068	1.174	0.105	0.05	0.059	0.130	0.962	New Proposal for C_{v2}

7. Conclusions

The design methods for predicting the shear strength of welded stainless steel I-shaped members were studied in this paper. Available research data of shear buckling tests on stainless steel I-shaped plate girders were collected from published literatures and were further utilised to assess the existing shear design methods for structural stainless steel, including the American AISC Design Guide 27, which refers to the ANSI/AISC 360-16 for carbon steel, and Eurocode 3 Part 1.4. It was shown that the two methods in ANSI/AISC 360-16 could not provide accurate shear strength predictions for stainless steel members. The first method for webs without tension field action (TFA) led to conservative results for specimens with low h_w/t_w ratios, yet generated unsafe predictions for webs with intermediate values of the ratio h_w/t_w, due to the insufficient consideration of strain hardening effect and material nonlinearity of stainless steels. Though similar overestimated results were obtained by the second method with TFA, closer predictions were achieved than the first one for slender specimens. Moreover, the summarised test data and predicted results from provisions in EN 1993-1-4+A1 were found to be in reasonable agreement.

In view of the inaccuracy of applying the provisions in ANSI/AISC 360-16 to the shear design of welded stainless steel I-shaped members, two new design methods for predicting the shear strength have been developed in this paper, which employ the same format of the ANSI/AISC 360-16 expressions. The first proposed method for webs without TFA was derived by converting the equations in EN 1993-1-4+A1 for web panels with non-rigid end posts, which was further adjusted to match the summarised test data. Improved and slightly conservative results were obtained by the newly proposed method, and the resistance factors of 1.163 and 1.019 were calculated by reliability analysis for the austenitic and duplex stainless steels, respectively, justifying the value of 0.9 recommended in the AISC Design Guide 27. The second proposed method considering TFA for interior webs with aspect ratios lower than or equal to 3.0 was achieved by developing new expressions for the critical shear buckling stress of stainless steel webs. Upon substituting the proposed expressions into ANSI/AISC 360-16 formulae with TFA, the predicted results were in satisfactory agreement with the test data points with resistance factors of 1.212 and 0.962 for austenitic and duplex stainless steel members, respectively. These two newly proposed methods are therefore recommended as alternative design approaches for future revisions of the AISC Design Guide 27 for structural stainless steel.

Acknowledgements

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