The Local Buckling Strength of Partially Stiffened Type 3CR12 Stainless Steel Compression Elements in Beam Flanges

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by

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SYNOPSIS

In this study the effect of the non-linear behaviour of stainless steels on the local buckling strength of partially stiffened compression elements in beam flanges is studied. The steel under investigation is Type 3CR12 stainless steel. Lipped channels were placed back to back to form a doubly symmetric lipped I-section. The different plasticity reduction factors suggested in the ASCE[3] and South African[13] stainless steel design specification for stiffened and unstiffened compression elements are used to compare experimental results with theoretical predictions.

It is concluded that the ASCE[3] and South African[13] stainless steel design specifications overestimate the local buckling stress in the beam flanges as well as the ultimate strength of partially stiffened stainless steel beams. The experimental results compare well with the theoretical predictions when the two plasticity reduction factors are used.

General Remarks

In contrast to carbon steels, stainless steels yield gradually under load. Due to the non-linear stress-strain relationship of stainless steels the design specifications for carbon and low alloy steels cannot be used. It is thus necessary to develop separate design criteria for stainless steels. For the overall stability of members the ASCE[3] and South African[13] stainless steel design specifications make use of plasticity reduction factors for design in the inelastic stress range. For overall stability the initial elastic modulus is replaced by the tangent modulus.

Johnson[9] and Wang[16] investigated the stability of stainless steel stiffened and unstiffened compression elements. Based on their work the ASCE[3] design specification for stainless steel structural members recommended that certain plasticity reduction factors could be used but that the effective width of these elements could be determined without using any plasticity reduction factors. It was found that this was in good agreement with experimental results.

No work was done at that stage on the stability of partially stiffened stainless steel compression elements. In this study the effect of the non-linear behaviour of stainless steels on the stability of partially stiffened compression elements in beam flanges is studied. The plasticity reduction factors recommended by Johnson[9] and Wang[16] will be used to determine the validity of their application to determine the effective width of partially stiffened compression elements. In this study it was decided to test doubly symmetric I-section beams.

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THEORETICAL MODEL

Mechanical Properties

The average stress-strain curves can be drawn by using the Ramberg-Osgood[10] equation as revised by Hill[8]. A detailed discussion of the Ramberg-Osgood[10] equation is given in Reference 14 and 15. The revised equation is given by Equation 1.

$$\varepsilon = \frac{f}{E_o} + 0.002\left(\frac{f}{f_y}\right)^n \qquad \text{Eq 1}$$

where

$$n = \frac{\log \frac{\varepsilon_y}{\varepsilon_p}}{\log \frac{f_y}{f_p}} \qquad \text{Eq 2}$$

ε strain
f stress
E_o initial elastic modulus
f_y yield strength
f_p proportional limit
n constant

The tangent modulus, E_t, is defined as the slope of the stress-strain curve at each value of stress. It is obtained as the inverse of the first derivative with respect to strain and can be computed by using Equation 3.

$$E_t = \frac{f_y E_o}{f_y + 0.002 n E_o \left(\frac{f}{f_y}\right)^{n-1}} \qquad \text{Eq 3}$$

The secant modulus, E_s, is defined as the stress to strain ratio at each value of stress and can be computed by using Equation 4.

$$E_s = \frac{E_o}{1 + 0.002 E_o \frac{f^{n-1}}{f_y^n}} \qquad \text{Eq 4}$$

Critical Local Buckling

The small deflection theory for the equilibrium of plates can be used to calculate the critical local buckling stress of a stiffened, unstiffened or partially stiffened compression element. Many researchers have suggested different approximate or more exact theories to calculate the critical local buckling stress in the inelastic stress range. Equation 5 can be used to calculate the critical local buckling stress for an isotropic plate in the inelastic stress range.

$$f_{cr} = \frac{\eta k \pi^2 E_o}{12(1-\nu^2)(w/t)^2} \qquad \text{Eq 5}$$

where

f_{cr}	critical local buckling stress
η	plasticity reduction factor
k	buckling coefficient
E_o	initial elastic modulus
ν	Poisson's ratio
w	flat width of the element
t	thickness of the element

The buckling coefficient k depends upon the edge rotational restraint, the type of loading and the aspect ratio of the plate. The buckling coefficients for the different plate elements under consideration can be summarised as follows.

$k = 0.425$	for unstiffened compression elements
$k = 4$	for stiffened compression elements
$0.425 < k < 4$	for partially stiffened compression elements

Several theories have been developed for the determination of the plasticity reduction factors for different types of compression elements. Johnson[9] and Wang[16] showed the validity of the following plasticity reduction factors for the determination of the critical local buckling stress for stainless steel structural members.

$\eta = 1$	for elastic buckling for carbon steel compression members
$\eta = E_S / E_o$	for buckling of unstiffened compression elements
$\eta = \sqrt{E_t / E_O}$	for buckling of stiffened compression elements

The above three plasticity reduction factors will be used in this study to compare the theoretical predictions with the experimental critical local buckling stresses.

Post buckling

For the theoretical calculation of the post buckling strength of partially stiffened compression elements the model suggested by the Canadian[6] and South African[12] carbon steel cold-formed design specifications, which is similar to the ASCE[3] stainless steel specification, will be used. The proposed South African[13] stainless steel design specification is similar. The equations in the above specifications will be revised to take into account the non-linear behaviour of stainless steels in the inelastic stress range by introducing plasticity reduction factors. The procedures described in the

South African[12] and Canadian[6] carbon steel cold-formed design specifications and the proposed South African[13] stainless steel design specification will be followed.

The design procedure to calculate the effective width of partially stiffened compression elements are divided into three categories. Case 1 deals with compression flanges that is fully effective, even if it has no lip and it is an unstiffened compression element. For this case it is not necessary to add a stiffener lip to the one side of the compression flange. The effective area of the compression flange is thus equal to the full unreduced area of the compression element. Only the stiffener lip has to be checked for local buckling. Figure 1 gives a general layout of partially stiffened compression elements.

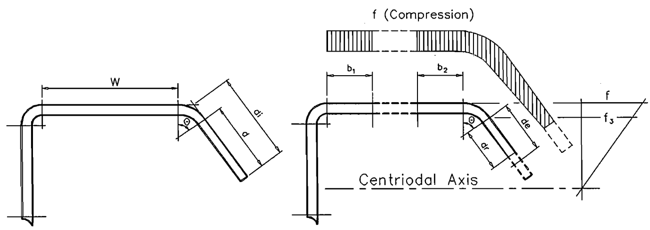

Figure 1 Typical Partially Stiffened Compression Element

The following equations are used in all the cases.

$$W_{\lim 1} = 0.644\sqrt{\frac{\eta k E_o}{f}} \quad \text{with } k = 0.425 \qquad \text{Eq 6}$$

$$W_{\lim 2} = 0.644\sqrt{\frac{\eta k E_o}{f}} \quad \text{with } k = 4 \qquad \text{Eq 7}$$

$$B = 0.95\sqrt{\frac{\eta k E_o}{f}}\left(1 - \frac{0.208}{W}\sqrt{\frac{\eta k E_o}{f}}\right) \qquad \text{Eq 8}$$

Case 1: $W \le W_{\lim 1}$

$B = W$
$d_r = d_e$

Case 2 $W_{\lim 1} < W \le W_{\lim 2}$

$$I_S = \frac{1}{12} t d^3 \sin^2\theta \qquad \text{Eq 9}$$

$$I_a = 400t^4\left[\frac{W}{W_{\lim 2}} - \sqrt{\frac{0.425}{4}}\right]^3 \qquad \text{Eq 10}$$

$$I_r = \frac{I_s}{I_a} \qquad \text{Eq 11}$$

$$d_r = d_e I_r \le d_e \qquad \text{Eq 12}$$

Case 3 W > W_{lim2}

$$I_a = t^4\left[115\frac{W}{W_{\lim 2}} + 5\right] \qquad \text{Eq 13}$$

For Cases 1 to 3 the value of the buckling coefficient k is calculated from the equations given in Table 1. The value for *n* is recommended by Schuster[7].

where

- W_{lim1} the limit for the flat width ratio above which an unstiffened compression element will buckle
- W_{lim2} the limit for the flat width ratio above which a stiffened compression element will buckle
- η plasticity reduction factor
- k buckling coefficient for different types of compression elements
- E_o initial elastic modulus
- f maximum stress in the compression element
- B effective width ratio b/t for compression elements
- W flat width ratio w/t for compression elements
- b effective width for compression elements
- w flat width of compression elements
- t thickness of steel
- d_e effective width of the stiffener
- d_r reduced effective width of the stiffener

Table 1 Values for Buckling Coefficient k

Case	I_r	$d_i/w \le 0.25$	$0.25 < d_i/w \le 0.8$
1		$k = 4$	$k = 4$
2 and 3	$I_r \ge 1$	$k = 4$	$k = 5.25 - 5d_i / w$
	$I_r < 1$	$k = 3.57I_r^n + 0.43$	$k = \left[4.82 - 5d_i / w\right]I_r^n + 0.43$
Note: $d/t \le 14$		$n = \frac{25}{43} - \frac{37W}{192}\sqrt{\frac{f}{\eta E_o}}$	$n \ge \frac{1}{3}$

EXPERIMENTAL PROCEDURE

Mechanical Properties

Uniaxial compression tests were carried out on specimens taken from the steel in the longitudinal directions. The tensile tests were carried out in accordance with the procedures outlined by the ASTM Standard A370-77[2].

Beam Tests

The average overall dimensions of the beams tested are given in Table 2. Short beams were tested to exclude the effect of lateral torsional buckling interaction. The beams are loaded statically and readings were taken every two seconds. The test is continued past the forming of local buckling waves until ultimate failure is reached when the load applied starts to decrease.

In this study the local and post buckling behaviour of lipped-I-sections were investigated. The different profiles were formed through a press brake process

The specimen cross-sections were proportioned in such a way to observe all the local buckling modes. In order to cover the whole range of variables governing element behaviour, the flat widths of the flanges were varied. The thickness of the sheet was 1.6 mm and the inside radius was 3 mm. The location of the strain gauges is shown in Figure 2. The placement of the strain gauges enables the detection of all the local buckling modes. The strain gauges mounted on the stiffener are used to indicate the presence of the local plate buckling mode. The strain gauges mounted on the flange are used to detect the flange stiffener buckling mode. The strain gauges mounted on the flange-stiffener junction are used to detect distortional buckling, which refers to the out-of-plane movement of the junction.

Table 2. Dimensions of Beams

No	t mm	A mm	B mm	C mm
120x30x20	1.6	120	30	20
120x40x20	1.6	120	40	20
120x50x20	1.6	120	50	20
120x60x20	1.6	120	60	20
120x70x20	1.6	120	70	20
120x80x20	1.6	120	80	20
120x90x20	1.6	120	90	20

The beams are tested in a four point loading setup arrangement as shown in Figure 2. The load is applied at a rate of less than 2 mm /minute movement of the crosshead of the Instron universal testing machine

DISCUSSION OF RESULTS

Mechanical Properties

Type 3CR12 steel yields gradually under load. This is in contrast to carbon and low alloy steels for which the transition to yielding is clearly noticeable. The mechanical properties of the stainless steel in longitudinal compression are.

E_o = 198 GPa
F_y = 326 MPa
F_p = 232 MPa

Beam Tests

The experimental results and theoretical predictions using the three plasticity reduction factors for the critical local buckling stresses and the ultimate strengths for the beams are given in Tables 3 and 4 and in Figures 3 and 4. The experimental critical local buckling stresses are compared with the theoretical predicted local buckling stresses by using the plasticity reduction factors in Equation 14.

$$\eta = 1$$
$$\eta = E_S / E_o \qquad \text{Eq 14}$$
$$\eta = \sqrt{E_t / E_o}$$

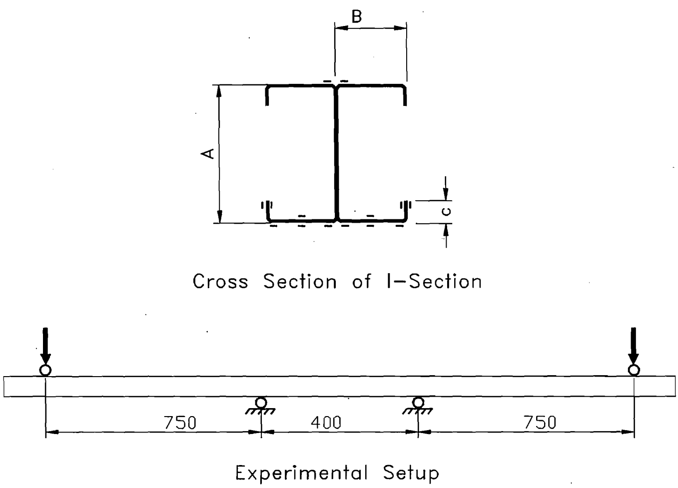

Figure 2 Dimensions and Detail of I-Sections

From Figure 3 it can be seen that the theoretical prediction using no plasticity reduction factor as recommended by the AISI[1], South African[12] and Canadian[6] carbon steel as well as the ASCE[3] and South African[13] stainless steel design specifications overestimates the critical local buckling stresses. The theoretical predictions using the other two plasticity reduction factors compare well with the experimental results. The following symbols apply in Table 3.

- f_e experimental critical local buckling stress
- f_{cr} theoretical critical local buckling stress using no plasticity reduction factor
- f_s theoretical critical local buckling stress using the secant plasticity reduction factor approach
- f_t theoretical critical local buckling stress using the tangent plasticity reduction factor approach

Table 3 Experimental and Theoretical Critical Local Buckling Results

Beam	Critical Local Buckling								
	w/t	d_i/w	f_e MPa	f_{cr} MPa	f_s MPa	f_t MPa	f_e/f_{cr}	f_e/f_s	f_e/f_t
No			Exper	Elastic	Secant	Tangent			
120x30x20	14.8	0.85	277(285)	833	335	271	0.33	0.83	1.02
120x40x20*	21.0	0.60	291(282)	855	336	275	0.34	0.87	1.06
120x50x20*	27.3	0.46	- (296)	638	319	261	-	-	-
120x60x20*	33.5	0.37	275(275)	474	298	247	0.58	0.92	1.11
120x70x20*	39.8	0.31	281(279)	362	275	232	0.78	1.02	1.21
120x80x20*	46.0	0.27	258(258)	283	248	215	0.91	1.04	1.20
120x90x20*	52.3	0.24	239(244)	225	215	192	1.06	1.11	1.24
* Slight distortional buckling found					Mean		0.67	0.97	1.14
( ) Distortional Buckling Stresses					COV		45.2	11.3	7.9

Table 4 Experimental and Theoretical Ultimate Results

Beam	Ultimate Strength								
	w/t	d_i/w	M_{exp} kNm	Me kNm	M_s kNm	M_t kNm	M_{exp}/M_e	M_{exp}/M_s	M_{exp}/M_t
No			Exper	Elastic	Secant	Tangent			
120x30x20	14.8	0.85	7.1*	9.1	7.9	7.3	0.78	0.90	0.97
120x40x20	21.0	0.60	8.7	10.4	8.6	7.5	0.84	1.01	1.16
120x50x20	27.3	0.46	9.8	11.6	8.8	7.7	0.84	1.11	1.27
120x60x20	33.5	0.37	10.0	11.4	8.9	7.9	0.88	1.12	1.27
120x70x20	39.8	0.31	10.3	11.5	9.1	8.0	0.90	1.13	1.29
120x80x20	46.0	0.27	10.2	11.7	9.2	8.1	0.87	1.11	1.26
120x90x20	52.3	0.24	10.6	11.7	9.2	8.2	0.91	1.15	1.29
*Support failed premature					Mean		0.86	1.08	1.22
					COV		5.15	8.31	9.64

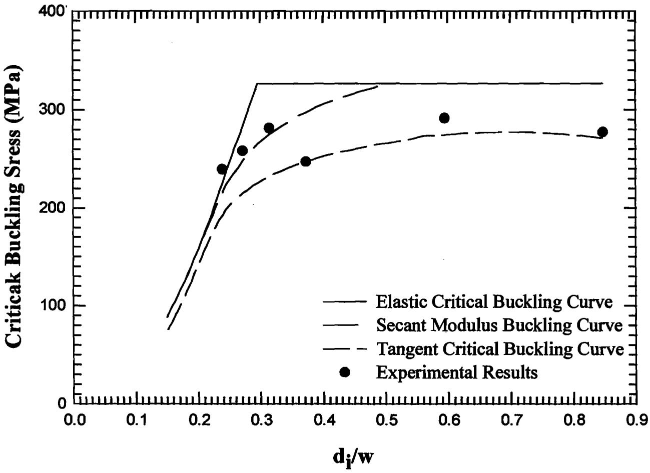

Figure 3 Critical Local Buckling Strength of Sections

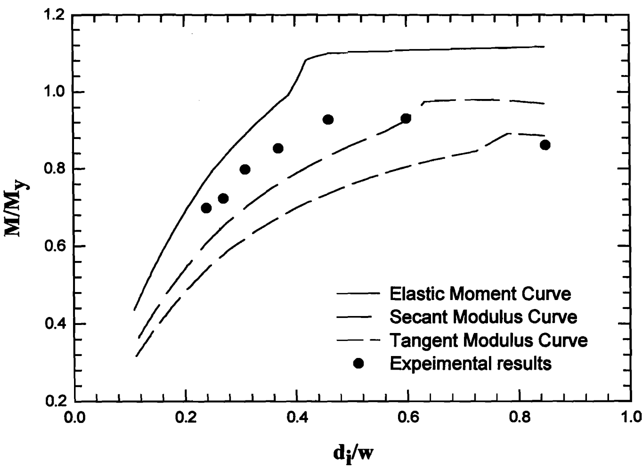

Figure 4 Ultimate Strength of Sections

The experimental results and theoretical prediction for the ultimate capacities of the beams are given in Table 4 and in Figure 4. From Table 4 it can be seen that the theoretical predictions using the secant and tangent modulus approach plasticity reduction factors are generally in better agreement with the experimental results. In a study by Buitendag[4,5] and Reyneke[11] on the strength of partially stiffened stainless steel compression members similar results were obtained. The following symbols apply in Table 4.

M_{exp} experimental ultimate failure moment
M_e elastic failure moment
M_s theoretical moment using the secant approach plasticity reduction factor
M_t theoretical moment using the tangent approach plasticity reduction factor

CONCLUSIONS

It is concluded in this study that the ASCE[3] and South African[13] stainless steel design specifications overestimate the local buckling stress as well as the ultimate strength of partially stiffened stainless steel compression elements. The experimental results compare well with the theoretical predictions when the two plasticity reduction factors are used.

From the limited number of tests carried out in this study it can be concluded that there is a general tendency for the elastic theory to overestimate the ultimate capacity of a section. For determining the ultimate capacity of a partially stiffened beam section the plasticity reduction factor using the secant modulus approach are in better agreement with the experimental results than the tangent modulus approuch.

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