Full slenderness range DSM approach for stainless steel hollow cross-section columns and beam-columns

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ABSTRACT

The behaviour of austenitic, ferritic and duplex stainless steel Rectangular and Square Hollow Section members subjected to compression and combined loading is investigated in this paper. A full slenderness range Direct Strength Method (DSM) approach is proposed based on experimental results and numerical strengths obtained from FE parametric studies. The method accounts for local buckling effects and enhanced material properties are also incorporated for those members stable enough to allow partial yielding of the cross-sections. The proposed method is based on strength curves previously provided for cross-sections although additional limitations have been adopted. The DSM approach for columns is based on existing buckling curves and provides accurate resistance predictions for slender and stocky cross-sections. The proposed DSM approach for beam-columns also improves capacity predictions for stocky and slender cross-sections obtained from the traditional methods for different bending moment distributions. This is attributed to the fact that the beam-column behaviour is directly calculated with a unique strength curve, considering the member and section slendernesses based on the elastic instabilities of the section subjected to the actual stress distribution instead of calculating the compressive and flexural strengths independently and combining these through an interaction equation, as is the traditional uncoupled approach. Finally, a reliability study of the full slenderness range DSM approach is presented to determine resistance factors for the different stainless steel grades columns and beam-columns.

HIGHLIGHTS

- A full slenderness range DSM approach for stainless steel SHS and RHS columns and beam-columns is proposed
- The method considers strain hardening and local buckling effects and has been extended to beam-columns with non-uniform bending distribution
- The method is assessed for different buckling curves and interaction expressions
- The new approach has been statistically validated and provides more accurate results than traditional methods

KEYWORDS

beam-column; column; Direct Strength Method; flexural buckling; hollow section; interaction equation; local buckling; stainless steel

LIST OF SYMBOLS

Latin upper case letters	
A	Gross-section area
A_{eff}	Effective cross-section area
B	Width of the cross-section
C_m	Equivalent moment factor
C_y	Compression strain factor
D_1, D_2, D_3	Parameters for beam-column interaction
E	Young's modulus
E_t	Tangent modulus
L	Length of the member
M	Bending moment
M_{cr1}	Critical elastic local buckling moment
M_n	Bending moment capacity of the cross-section accounting for inelastic stress distribution
M_{ne}	Bending moment strength of fully effective members
M_{nl}	Nominal bending moment resistance
M_{ocr}	Bending moment causing local buckling under combined loading conditions
M_{on}	Bending moment strength causing failure under combined loading conditions
M_{one}	Bending moment strength of fully effective members in combined loading
M_p	Full plastic moment capacity
M_y	First yielding bending moment
P	Compression axial load
P_{AS}	Predicted flexural buckling resistance using the AS/NZS4673 buckling curves
P_{cre}	Minimum of the elastic flexural, torsional and flexural-torsional buckling load
P_{cr1}	Critical elastic local buckling load under compression
$P_{DSM,AS}$	Predicted flexural buckling resistance using the DSM approach with buckling curves in AS/NZS4673

$P_{DSM,EN}$	Predicted flexural buckling resistance using the DSM approach with buckling curves in EN1993-1-4
P_{EN}	Predicted flexural buckling resistance using EN1993-1-4 buckling curves
P_{enh_ne}	Flexural buckling resistance of columns accounting for enhanced material properties
P_{ne}	Flexural buckling resistance of the fully effective cross-section
P_{nl}	Cross-section or member resistance under compression
P_{ocr}	Compression load causing local buckling under combined loading conditions
P_{on}	Compression strength causing failure under combined loading conditions
P_{one}	Compression strength of fully effective members in combined loading
$P_{u,exp}, P_{u,FE}$	Experimental or numerical column strength
P_y	Cross-section squash load
R_0	Gross-section capacity of the cross-section or member
R_{crl}	Critical elastic local buckling resistance
R_{enh_nl}	Nominal resistance of the cross-section or member accounting for enhanced material properties
R_{nl}	Nominal resistance of the cross-section or member

Latin lower case letters	
$d_{u,exp}, d_{u,FE}$	Experimental or numerical lateral deflections
e	Load eccentricity
e_{eq}	Equivalent load eccentricity
k	Interaction factor for beam-column checks
m	Second strain hardening exponent
n	First strain hardening exponent
$r_{AS/NZS}$	Predicted beam-column resistance parameter using the DSM approach interaction factor in AS/NZS4673
r_{crl}	Local buckling behaviour parameter for beam-columns
r_{EN1-1}	Predicted beam-column resistance parameter using the DSM approach interaction factor in EN1993-1-1
r_{EN1-4}	Predicted beam-column resistance parameter using the DSM approach interaction factor in EN1993-1-4
r_{enh_nl}	Nominal resistance parameter for beam-columns accounting for enhanced material properties
r_{ne}	Resistance parameter corresponding to fully effective member strength
r_u	Resistance parameter of experimental and numerical beam-columns
r_y	Resistance parameter corresponding to the yielding of cross-sections subjected to combined loading
r_{Zhao}	Predicted beam-column resistance parameter using the DSM approach interaction factor by Zhao et al. [35]

Greek lower case letters	
α	Imperfection factor for flexural buckling
β	Factor for flexural buckling in AS/NZS4673; reliability index
β_0	Target reliability index
χ	Reduction factor for flexural buckling
ϵ_u	Ultimate strain
ϵ_y	Yield strain
ϕ	Value to determine the reduction factor χ
ϕ_c	Resistance factor for columns
γ_{M1}	Partial safety factor for resistance of members to instability
λ_0	Limiting slenderness for flexural buckling

λ_l	Factor for flexural buckling in AS/NZS4673
λ_c	Member slenderness
λ_l	Cross-section/local slenderness
λ_{lim}	Limiting slenderness for local buckling in DSM
λ_n	Generalized slenderness for beam-columns
ρ	Reduction factor for local buckling in EWM
$\sigma_{0.2}$	Proof stress corresponding to the 0.2% plastic strain
σ_{enh_nl}	Stress at which the column fails
σ_u	Tensile strength
ψ	Ratio of moments in segment

1. INTRODUCTION

The development of accurate and simple design guidance for stainless steel structures is essential for the increased use of this corrosion resistant material, which is also characterized by excellent mechanical properties, excellent strength retention at elevated temperature and pleasing aesthetic appearance. Since most stainless steel grades are characterized by a nonlinear stress-strain relationship with a pronounced gradual yielding, efficient strength and stability design provisions need to include strain hardening effects instead of directly extending the expressions codified for carbon steel structures which are premised on steels with an extended yield plateau.

Commonly used in construction as cold-formed elements, stainless steel structures exhibit slender cross-sections subjected to local and distortional buckling modes. Stainless steel standards (e.g. EN1993-1-4 [1], AS/NZS4673 [2], SEI/ASCE 8-02 [3]) have traditionally accounted for these buckling effects by using the effective width method, resulting in tedious and iterative calculations. Instability effects can be easily considered through the strength curves in the Direct Strength Method (DSM), developed by Schafer and Pekoz [4], with the use of software to determine elastic buckling loads. Although the method has been implemented in the North American AISI-S100-12 Specification [5] for carbon steel structures and new strength curves have been proposed to adapt the DSM to stainless steel cross-sections (Becque et al. [6], Niu et al. [7], Huang and Young [8]), the DSM has not yet been included in stainless steel standards. Rossi and Rasmussen [9] proposed a full slenderness range approach which considers strain-hardening effects in the DSM formulation for stainless steel sections in compression, improving the capacity predictions for stocky cross-sections. Arrayago et al. [11] demonstrated the

applicability of this approach for different loading conditions such as bending and combined loading. Based on the DSM approach proposed by Rasmussen [10] for beam-columns, the full slenderness range approach was also extended to the combined loading behaviour of stainless steel Square and Rectangular Hollow Sections (SHS and RHS) cross-sections by [11].

The study presented in this paper is an extension of the research work presented in [11] for SHS and RHS cross-sections to austenitic, ferritic and duplex stainless steel members subjected to compression and combined loading. The extension from section capacity to member capacity relies on an extensive experimental database and additional finite element parametric studies. The full slenderness range DSM approach is evaluated for members in compression and the applicability of the method proposed in [10] for beam-columns is also assessed. Results are compared with those obtained using the design provisions based on the effective width method and the most appropriate buckling curves and interaction expressions are identified.

2. DIRECT STRENGTH METHOD

The Direct Strength Method (DSM) is a design procedure developed by Schafer and Pekoz [4] which allows the consideration of local, distortional and member buckling effects in an easy and consistent manner as an alternative to effective width calculations. The method is based on strength curves which depend on the buckling slenderness for the three modes measuring the susceptibility of the member to local, distortional and member buckling, and uses software to determine elastic buckling loads. For SHS and RHS, only the local and member buckling modes are relevant. Although the DSM approach is currently included in the AISI-S100-12 Specification [5] for cold-formed carbon steel structures, it has not yet been included in stainless steel standards.

2.1 STRENGTH CURVES FOR ENHANCED MATERIAL PROPERTIES AND LOCAL BUCKLING

The Direct Strength Method considers the effect of local buckling by reducing the resistance of the gross section through a simple strength curve depending on the slenderness of the cross-section λ_1 . According to the AISI-S100-12 Specification [5], the strength curve codified for carbon steel members can be expressed in the general form of Eq. (1), where the nominal resistance of the cross-section R_{nl} is calculated by reducing the gross-section capacity R_0 due to the effect of local buckling as a function of the local slenderness calculated from Eq. (2).

$$\frac{R_{nl}}{R_0} = \begin{cases} 1 & \text{for } \lambda_1 \leq 0.776 \\ \frac{1}{\lambda_1^{0.8}} - \frac{0.15}{\lambda_1^{1.6}} & \text{for } \lambda_1 > 0.776 \end{cases} \quad (1)$$

$$\lambda_1 = \sqrt{\frac{R_0}{R_{cr}}} \quad (2)$$

Depending on the on the loading case, nominal resistances R_{nl} need to be predicted considering different gross-section R_0 resistances and critical elastic local buckling resistances R_{cr} , which can be obtained from a number of numerical methods and related software programs based on finite element and finite strip methods. When the cross-sectional compression resistance P_{nl} is evaluated, R_0 corresponds to the cross-section squash load P_y and R_{cr} is the critical elastic local buckling load P_{cr} . Alternatively, for the evaluation of the cross-sectional bending moment resistance, M_{nl} , R_0 and R_{cr} equal the moment of first yield M_y and the critical elastic local buckling moment M_{cr} respectively. Finally, the flexural buckling resistance is obtained considering R_0 equal to the flexural buckling resistance of the fully effective cross-section for the considered buckling curve P_{ne} and the critical elastic local buckling load P_{cr} .

Becque et al. [6] investigated the local buckling behaviour of stainless steel structures through an experimental programme on lipped channel section and I-section columns, complemented by comprehensive numerical studies on SHS and RHS columns. Although no differences were appreciated among the different stainless steel grades, lipped channel and I-section results highlighted the necessity of an alternative strength curve for stainless steels. This curve is presented in Eq. (3) and it is slightly lower than that given for carbon steel sections, with a lower limiting slenderness equal to 0.474. Niu et al. [7] studied stainless steel cross-sections subjected

to bending moment and concluded that the local buckling reduction can be also conservatively calculated from Eq. (3), thus producing a unique strength curve for both compression and bending governing the local buckling strength of stainless steel cross-sections.

$$\frac{R_{nl}}{R_0} = \begin{cases} 1 & \text{for } \lambda_1 \leq 0.474 \\ \frac{0.95}{\lambda_1^{0.8}} - \frac{0.22}{\lambda_1^{1.6}} & \text{for } \lambda_1 > 0.474 \end{cases} \quad (3)$$

Recent research by Rossi and Rasmussen [9] and Arrayago et al. [11] on stainless steel SHS and RHS subjected to different loading conditions produced an accurate full slenderness range DSM approach for cross-sectional compression, bending and combined loading resistances by including the effect of strain hardening into the DSM formulation. The approach is given in Eq. (4) and was originally proposed [9] for compression and distortional buckling effects at cross-sectional level. The method was extended to bending and combined loading [11] and adapted to the carbon steel strength curve, as given in Eq. (5). These approaches adopt a linear relationship between the resistance and the slenderness in the low slenderness range, instead of the classical horizontal yield limit. The predicted capacity-to-conventional yield limit ratio tends to $\sigma_u/\sigma_{0.2}$ as the slenderness approaches zero in a linear variation. The final R_{enh_nl} strength curves presented in Eq. (4) and Eq. (5) consider both strain hardening and local buckling effects and are based on the original stainless steel and carbon steel strength curves for local buckling, respectively.

$$\frac{R_{enh_nl}}{R_0} = \begin{cases} 1 + (1 - 2.11\lambda_1) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1 \right) & \text{for } \lambda_1 \leq 0.474 \\ \frac{0.95}{\lambda_1^{0.8}} - \frac{0.22}{\lambda_1^{1.6}} & \text{for } \lambda_1 > 0.474 \end{cases} \quad (4)$$

$$\frac{R_{enh_nl}}{R_0} = \begin{cases} 1 + (1 - 1.29\lambda_1) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1 \right) & \text{for } \lambda_1 \leq 0.776 \\ \frac{1}{\lambda_1^{0.8}} - \frac{0.15}{\lambda_1^{1.6}} & \text{for } \lambda_1 > 0.776 \end{cases} \quad (5)$$

Nevertheless, it should be noted that the enhanced material property approach proposed in [9] for stainless steel columns is slightly different from that presented in Eq. (4), which was limited to cross-sectional behaviour. The method proposed in [9] for stainless steel columns was based on the buckling curves provided in AS/NZS4673 [2] and provides an expression that accounts for

strain hardening effects in the flexural buckling resistance P_{enh_ne} as a function of the member slenderness λ_c as per Eq. (6), where λ_{lim} is the limiting slenderness at which the reduction factor $\chi = 1/\left[\phi + \sqrt{\phi^2 - \lambda_c^2}\right]$ becomes equal to unity. For the overall-local buckling interaction in columns, the method in [9] adopts the Becque et al. [6] approach for stainless steel cross-sections given in Eq. (3) assuming $R_0 = P_{enh_ne}$, obtained from Eq. (6).

$$\frac{P_{enh_ne}}{P_y} = \begin{cases} 1 + \left(1 - \frac{\lambda_c}{\lambda_{lim}}\right) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1\right) & \lambda_c \leq \lambda_{lim} \\ \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}} & \lambda_c > \lambda_{lim} \end{cases} \quad (6)$$

2.2 INELASTIC RESERVE STRENGTH IN BENDING

Stocky SHS and RHS beams have significant inelastic reserve strength, particularly for stainless steel grades, and ultimate bending moments commonly exceed the full plastic moment capacity M_p . This is usually incorporated in design standards, since EN1993-1-4 [1] assigns a full plastic capacity to cross-sections classified as Class 1 or 2 while AS/NZS4673 [2] also adopts the full plastic capacity for rectangular and square hollow sections. Provisions given in “Procedure II” of AS/NZS4673 [2] and SEI/ASCE 8-02 [3] allow the consideration of the inelastic reserve strength by adopting a compression strain factor C_y to determine the maximum compressive strain and assuming an ideally elastic-plastic stress-strain curve throughout the cross-section.

The implementation of the inelastic reserve strength in the DSM formulation was conducted by Shifferaw and Schafer [12] for cold-formed carbon steel C- and Z-beams, and this approach was included in the revised AISI-S100-16 Specification [13]. According to [12], the inelastic reserve strength can be obtained from Eq. (7) as a function of the maximum compressive strain $C_y \cdot \epsilon_y$, where C_y is the compression strain factor calculated as a direct function of the cross-sectional slenderness λ_t . Although [12] proposed a predictive expression for the determination of the compression strain factor, recent research by Arrayago et al. [11] demonstrated that the expression was too conservative for stainless steel SHS and RHS beams. Alternatively, it was suggested to keep the C_y provisions from “Procedure II” given in [2] and [3] as shown in Eq. (8). The

maximum strain in inelastic bending was limited to $3\varepsilon_y$ for carbon steel and stainless steel cross-sections in order to be consistent with the scope of the different standards [5], [2] and [3].

$$M_n = M_y + (M_p - M_y) \left[1 - \frac{1}{C_y^2} \right] \quad (7)$$

$$C_y = \begin{cases} 3 & \lambda_1 \leq 0.58 \\ 3 - 2 \left(\frac{\lambda_1 - 0.58}{0.196} \right) & 0.58 \leq \lambda_1 \leq 0.776 \\ 1 & \lambda_1 \geq 0.776 \end{cases} \quad (8)$$

2.3 DSM APPROACH FOR COMBINED LOADING CONDITIONS

AISI-S100-12 [5] does not provide a DSM-based expression for the design of beam-columns so the general interaction expressions need to be applied with the pure flexural buckling and bending resistances calculated using the DSM approaches presented in previous sections. However, Rasmussen [10] extended the existing DSM approach for flexural buckling to beam-columns by introducing resistance parameters defined as radial distances in the M/M_y - P/P_y plane. In this approach the beam-column behaviour is directly considered with a unique strength curve, calculating the member and cross-section slendernesses based on the elastic instabilities of the section subjected to the actual stress distribution.

According to [10] the DSM approach for beam-columns first considers the member behaviour through an interaction expression as given in Eq. (9) from which the overall buckling strengths P_{one} and M_{one} are obtained. Bending moments are expressed in terms of the axial load through a load eccentricity e , ($M_{one} = e \cdot P_{one}$), and the resistance parameter r_{ne} corresponding to the member strength can be obtained through Eq. (10). The method allows different interaction factors k for beam-columns and, several equivalent moment factors C_m , to be considered through Eq. (9), in which P_{ne} is the flexural buckling resistance of the fully effective column and M_{ne} is the bending moment capacity of the fully effective member.

$$\frac{P_{one}}{P_{ne}} + k \cdot C_m \frac{M_{one}}{M_{ne}} = 1 \quad (9)$$

$$r_{ne} = \sqrt{\left(\frac{P_{one}}{P_y} \right)^2 + \left(\frac{M_{one}}{M_y} \right)^2} \quad (10)$$

The local buckling behaviour of the cross-sections is introduced using the r_{cr1} parameter defined in Eq. (11), where P_{ocr} and M_{ocr} are the axial compression and moment causing local buckling under combined compression and bending loading conditions, obtained from a buckling analysis. Finally, a generalized local slenderness is calculated from Eq. (12).

$$r_{cr1} = \sqrt{\left(\frac{P_{ocr}}{P_y}\right)^2 + \left(\frac{M_{ocr}}{M_y}\right)^2} \quad (11)$$

$$\lambda_n = \sqrt{\frac{r_{ne}}{r_{cr1}}} \quad (12)$$

Assuming that the strength equations derived for cross-sectional and column behaviour are also applicable to beam-columns, the nominal resistance parameter r_{enh_nl} can be calculated through Eqs (4) and (5) by adopting $R_{enh_nl}=r_{enh_nl}$, $R_0=r_{ne}$ and $\lambda_l=\lambda_n$. The predicted axial compression and bending strengths of the member P_{on} and M_{on} are finally obtained from Eq. (13).

$$r_{enh_nl} = \sqrt{\left(\frac{P_{on}}{P_y}\right)^2 + \left(\frac{M_{on}}{M_y}\right)^2} \quad (13)$$

This DSM-based approach for beam-columns was modified and adapted to cross-sections subjected to combined compression and uniaxial bending loading conditions in Arrayago et al. [11] by considering that the radial distance governing the member strength changes to the yield radial distance r_y when cross-sections subjected to combined loading are considered. Arrayago et al. [11] concluded that although the linear interaction expression was accurate for slender cross-sections, the interaction approach given in EN1993-1-1 [14] for Class 1 and 2 cross-sections was more adequate when stocky cross-sections were analysed.

3. AVAILABLE EXPERIMENTAL DATA AND FE ULTIMATE STRENGTHS

The full slenderness range DSM approaches presented in this paper have been investigated through an extensive experimental and numerical database of stainless steel SHS and RHS members subjected to compression and combined loading. This section presents the database of experimental and numerical strengths and summarizes the most relevant aspects of the conducted Finite Element (FE) numerical analyses.

3.1 EXPERIMENTAL DATA COLLECTION

The behaviour of stainless steel SHS and RHS members subjected to compression and combined loading has been analysed through different experimental programmes over the last three decades, covering austenitic, ferritic, duplex and lean duplex grades. In the present study, the austenitic stainless steel column tests reported in [15, 17-21] have been considered, while the considered tests on ferritic and duplex grades correspond to those reported in [22-24] and [25-28]. For beam-columns, the experimental programmes reported in [15, 29-30], [16, 23-24, 29] and [28, 31] corresponding to austenitic, ferritic and duplex or lean duplex grades respectively have been included in the analysis. Most of the tests were conducted for bending about the minor axis and under pin-ended conditions but major axis bending was analysed in [15]. The tested bending moment distributions in beam-column specimens generally corresponded to constant bending moment distributions, although [16] reported some tests under bending moment gradient.

3.2 FE MODEL VALIDATION AND PARAMETRIC STUDIES

The column and beam-column FE models utilized in the parametric study were created by the general purpose software Abaqus and were first validated against the experimental results on ferritic stainless steel members reported in Arrayago et al. [23]. The mid-surfaces of the cross-sections were modelled using the Abaqus four-node shell element with reduced integration S4R, widely used for modelling cold-formed stainless steel cross-sections. The computational efficiency and reliability of the results was ensured by a mesh convergence study and by limiting the length of the shell elements to 5 mm. Initial geometric imperfections were introduced in the models in the shape of the first elastic buckling mode obtained from a linear buckling analysis, with imperfection amplitudes equal to those measured in the specimens and reported in [23] for the verification of the FE model. The nonlinear analyses were performed by using a modified Riks method.

Pin-ended boundary conditions were modelled by kinematic couples at the ends of the member connecting the edge elements to two reference points defined according to the distance between knife-edges reported in [23], 50mm away from each specimen end. All degrees of freedom except the rotation about the minor axis were restrained at the lower reference point, while longitudinal

displacements and minor axis rotations were set free in the upper one. Loads were then introduced as imposed displacements at the upper reference point. For the beam-column models, combined loading conditions were introduced as eccentric loads by defining non-centred reference points.

Two different material definitions were considered during the validation of the FE models. First, the flat and corner regions were differentiated and the corresponding measured stress-strain properties were assigned to each region. Corner regions were extended also to the adjacent flat parts by a width equal to two times the thickness of the element, as assumed in [24,25]. Residual stresses were not explicitly introduced in the models since according to [21] the stress-strain curves obtained from coupon tests already include the bending residual stresses, and the membrane residual stresses were assumed to be negligible. Additionally, the different behaviour of flat and corner regions is considered by determining the weighted average material properties from the experimentally measured results where the same material properties were assigned to the entire cross-section to simplify the FE model as suggested by Hradil and Talja [32]. Parameters are weighted according to the area of the considered flat or corner region compared to the total area of the cross-section, assigning the value of the corresponding material parameter to each region. The material parameters corresponding to the flat and corner regions and the weighted average material properties can be found in [23].

Experimental load-lateral deflection curves have been compared to those obtained from FE analyses considering the two different material definitions: different constitutive laws in flat and corner regions (FE) and the weighted average material properties for the entire cross-section (FE, average material). This comparison is presented in Figure 1 for typical column and beam-column specimens. Table 1 reports the mean values and the coefficients of variation (COVs) of the numerical-to-experimental ultimate load ratios and the corresponding lateral deflections for the columns and beam-columns considered in the model validation. The results demonstrate that the developed numerical models are capable of accurately predicting the ultimate loads and replicating the full experimental load-deflection responses. It is also noteworthy that the failure modes of the obtained FE models are in good agreement with the experimental results.

Figure 1. Comparison of experimental and FE curves for ferritic stainless steel members in compression and combined loading.

Table 1. Comparison between experimental and FE results for columns and beam-columns.

Specimen	Flat and corner material		Weighted average material	
	$P_{u,FE}/P_{u,exp}$	$d_{u,FE}/d_{u,exp}$	$P_{u,FE-av}/P_{u,exp}$	$d_{u,FE-av}/d_{u,exp}$
80x80x4 – CC	1.00	0.93	1.04	1.07
80x80x4 – EC1	1.02	1.08	0.99	0.94
80x80x4 – EC2	1.02	1.04	0.99	1.05
60x60x3 – CC	1.02	0.99	1.03	0.88
60x60x3 – EC	1.01	1.02	1.02	1.02
80x40x4 – CC	1.01	1.03	1.02	0.85
80x40x4 – EC1	1.03	1.03	1.04	0.96
120x80x3 – CC	0.97	0.80	1.00	0.83
120x80x3 – EC1	0.99	0.87	0.99	1.02
70x50x2 – CC	1.02	1.00	1.02	1.00
70x50x2 – EC1	1.01	0.96	0.99	0.98
70x50x2 – EC2	1.04	1.03	1.05	1.02
Mean	1.01	0.98	1.01	0.97
COV	0.019	0.082	0.022	0.080

Once the numerical models were validated against experimental results for ferritic stainless steel SHS and RHS members subjected to compression and combined loading, parametric studies were conducted through a combination of Python scripts and Abaqus modelling tools. Since the measured initial global imperfection amplitudes reported in [23] ranged between $L/2000$ and $L/1000$, the average imperfection amplitude represented by $L/1500$ was considered in the

parametric studies, where L is the member length, in the shape of the first elastic buckling mode. Additionally, local initial imperfections were also introduced with imperfection amplitudes predicted from the modified Dawson and Walker [33] model proposed by Gardner and Nethercot [34].

The parametric studies were primarily focused on ferritic stainless steel, though analyses were also conducted for austenitic and duplex grades. The material properties were obtained from the results reported in Arrayago et al. [23] for ferritic grades, while those used in the parametric studies by Zhao et al. [35] were considered for austenitic and duplex stainless steels, which correspond to the ferritic, austenitic and duplex sub-classes EN1.4003, EN1.4404 and EN1.4162. In all analyses, the weighted average material properties were assigned to the entire cross-section, which were calculated from the material properties of the flat and corner regions of the cross-sections reported as explained before for each stainless steel grade. The relevant material parameters are reported in Table 2 where E is the Young's modulus, $\sigma_{0.2}$ is the proof stress corresponding to the 0.2% plastic strain, σ_u is the tensile strength and ε_u is the corresponding ultimate strain. Strain hardening exponents n and m are also provided. The stress-strain curves used for the numerical analyses were obtained using the stress-strain curve formulation presented in Arrayago et al. [36] in combination with the parameters shown in Table 2.

Table 2. Material parameter definition for parametric studies.

Stainless steel	E [GPa]	$\sigma_{0.2}$ [MPa]	σ_u [MPa]	ε_u [%]	n	m
Austenitic	197.8	417	651	35.9	5.5	3.7
Ferritic	185.7	490	533	4.8	11.0	3.2
Duplex	201.3	707	874	19.1	5.6	4.9

The column behaviour was analysed using SHS cross-sections with outer widths ranging from 60mm to 200mm, while for RHS the widths varied between 60mm and 100mm with aspect ratios equal to 1.2, 1.5 and 2, and thicknesses ranging between 2mm and 6mm. For ferritic stainless steels, each cross-section was modelled for 12 different column slendernesses, ranging between 0.25-3.0 and leading to more than 1200 FE models. For austenitic and duplex grades, the conducted parametric studies were more limited, using 160 models for each grade. Regarding

beam-columns, the minor axis strength was analysed for 30 cross-sections considering three different bending moment distributions (uniform, triangular and bitriangular) with several variations of the load eccentricity. Similar cross-section geometries to those defined for the column parametric studies were considered, considering four member slendernesses per cross-section and leading to more than 1350 FE models for ferritic stainless steels and 430 models for the austenitic and duplex grades. Several load eccentricities e providing different compression-to-bending ratios were considered. The adopted e/B ratios were 0.1, 0.3, 0.75 and 1.5, where B is the width of the cross-section in the same plane as the eccentricity (e). Since the majority of the available experimental results were tested under uniform bending moment distributions plus compression loads, bending moment diagrams corresponding to triangular ($\psi=0$) and bitriangular ($\psi=-1$) distributions were considered in the parametric studies in addition to the uniform distribution ($\psi=1$). Here, ψ is the ratio of the smaller to the larger moment at member ends, and is negative when the member is bent in reverse curvature and positive when bent in single curvature, according to the EN1993-1-1 [14] definition.

4. FULL SLENDERNESS RANGE DSM APPROACH FOR MEMBERS IN COMPRESSION

This section presents a full slenderness range DSM approach for stainless steel SHS and RHS columns. Results obtained using the new approach are compared with those corresponding to the effective width method for the same buckling curves in order to identify the most accurate and appropriate design method.

4.1 FLEXURAL BUCKLING CURVES

The capacity of stainless steel columns is usually determined in design specifications through buckling curves which provide the strength reduction due to flexural buckling. However, the interaction with local buckling effects is considered in different manners depending on the adopted method. The most common standards (e.g. EN1993-1-4 [1], AS/NZS4673 [2] and SEI/ASCE 8-02 [3]) first evaluate the effect of local buckling through the Effective Width Method (EWM), defining a modified slenderness based on the effective cross-sectional area, after which

the overall buckling strength of the column is evaluated. Alternatively, the DSM approach first considers the overall buckling of the member with a fully effective cross-section and then introduces the resistance reduction due to local buckling, as described in Section 2.1. Therefore, the accuracy of both the EWM and DSM methodologies is highly dependent on the correct prediction of the flexural buckling strength of stainless steel columns.

According to EN1993-1-4 [1] and AS/NZS4673 [2], the flexural buckling resistance of stainless steel columns can be calculated from Eq. (14), where χ is the reduction factor due to flexural buckling defined in Eq. (15), A_{eff} is the effective area of the cross-section (the gross-section area A is considered for fully effective cross-sections) and γ_{M1} is the partial safety factor. The member slenderness λ_c can be calculated from $\lambda_c = \sqrt{P_y/P_{\text{cre}}}$ where P_{cre} corresponds to the elastic flexural buckling load of the column. AS/NZS4673 [2] considers a nonlinear expression for the generalized imperfection factor $\eta = \alpha((\lambda_c - \lambda_1)^\beta - \lambda_0)$ in Eq. (16) with six buckling curves for various stainless steel grades providing values for the different factors, while EN1993-1-4 [1] establishes that the buckling curve c should be considered for hollow sections, adopting $\eta = \alpha(\lambda_c - \lambda_0)$ for all stainless steel grades. AS/NZS4673 [2] also allows the use of the tangent modulus approach in the prediction of the flexural buckling resistance, where the nonlinear stress-strain response of the material is considered by allowing a gradual yielding through the use of the tangent modulus E_t corresponding to the buckling stress. However, since the calculation of this tangent modulus involves an iterative design process and the method does not consider member imperfections, the tangent modulus approach has not been analysed in this paper.

$$P_{\text{nl}} = \frac{\chi A_{\text{eff}} \sigma_{0.2}}{\gamma_{M1}} \quad (14)$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}} \leq 1.0 \quad (15)$$

$$\phi = 0.5(1 + \eta + \lambda_c^2) \quad (16)$$

4.2 FULL SLENDERNESS RANGE DSM APPROACH FOR STAINLESS STEEL COLUMNS

The assessment of the DSM approach for SHS and RHS columns is presented in Figures 2, 3 and 4 for austenitic, ferritic and duplex stainless steel grades respectively. Experimental and numerical column strengths P_u have been nondimensionalised with respect to the flexural buckling strengths of the fully effective cross-sections P_{ne} and plotted against the corresponding local slenderness $\lambda_1 = \sqrt{P_{ne}/P_{cr1}}$, where P_{cr1} corresponds to the elastic local buckling load. The flexural buckling strengths of the fully effective cross-sections P_{ne} have been obtained from the buckling curves provided in EN1993-1-4 [1] and AS/NZS4673 [2] and have been plotted with empty and solid markers, respectively, in Figures 2, 3 and 4. Slenderness values have been calculated from Eq. (2), in which the critical elastic local buckling load P_{cr1} was obtained from CUFSM (Schafer and Ádány [37]) calculations for pure compression. In addition, DSM strength curves for carbon steel cross-sections codified in AISI-S100-12 [5] (Eq. (1)) and proposed for stainless steel columns by Becque et al. [6] (Eq. (3)) have also been plotted for comparison.

Figure 2. Assessment of the DSM approach for austenitic stainless steel SHS and RHS members in compression for different buckling curves.

Figure 3. Assessment of the DSM approach for ferritic stainless steel SHS and RHS members in compression for different buckling curves.

Figure 4. Assessment of the DSM approach for duplex and lean duplex stainless steel SHS and RHS members in compression for different buckling curves.

Figures 2, 3 and 4 demonstrate that results corresponding to the buckling curves in AS/NZS4673 [2] and EN1993-1-4 [1] are similar although the P_u/P_{ne} ratios are slightly lower for EN1993-1-4 [1], particularly for austenitic and ferritic stainless steels, since the buckling curves in AS/NZS4673 [2] are considerably lower than the EN1993-1-4 [1] c -curve for these materials. For

duplex and lean duplex, both standards provide similar buckling curves and the differences are notably smaller. It is noticeable from Figures 2 to 4 that for a given buckling curve, no differences are appreciated in the overall-local buckling interaction among the different stainless steel grades. The results also highlight that the DSM approach provides good prediction of the ultimate capacity of stainless steel SHS and RHS columns with slender cross-sections and that the most accurate results are obtained for the strength curve provided for carbon steel cross-sections given in Eq. (1). Similar results were previously reported by Becque and Rasmussen [38] for tubular columns and in [11] regarding the cross-sectional behaviour of stainless steel SHS and RHS.

Figures 2-4 also highlight the existence of a considerable strength reserve at low values of slenderness (λ_i). The non-dimensionalised strengths well above the $P_u/P_{ne}=1$ line are for columns with low cross-section slenderness. This reserve can be partly attributed to the effect of strain hardening and as it can be noticed, the highest P_u/P_{ne} ratios are observed for austenitic stainless steel columns, followed by duplex and ferritics, as expected from the typical stress-strain behaviour of these grades. A few results with P_u/P_{ne} ratios lower than unity with low local slenderness can be appreciated in Figure 2 for austenitic members when the strength is nondimensionalised using the buckling curve codified in EN1993-1-4 [1], and in Figure 3 for ferritic stainless steel columns. These results do not correspond to an unsafe resistance prediction of the DSM approach, but to an overestimated flexural buckling capacity for the considered buckling curves.

The assessment of the DSM approach is presented in Table 3 for slender cross-sections, where the accuracy of the expressions considering the local buckling interaction in stainless steel SHS and RHS columns is analysed for the buckling curves provided in EN1993-1-4 [1] and AS/NZS4673 [2]. The mean values and COVs of the predicted-to-experimental (or FE) strengths obtained with the DSM are compared to those obtained through the codified Effective Width Methods for the same buckling curves in order to evaluate the accuracy of each method. P_{EN} represents the predicted resistance using the EN1993-1-4 [1] buckling curve, while P_{AS} corresponds to capacity predictions for the AS/NZS4673 [2] buckling curves. For the DSM

approach, strength curves corresponding to carbon steel (CS-curve) and stainless steel (SS-curve) cross-sections, given in Eqs. (1) and (3), are assessed. Cross-sections with a cross-sectional slenderness higher than $\lambda_1 \geq 0.776$ have been considered in the DSM approach for the CS-curve in Table 3, while those with a cross-sectional slenderness higher than $\lambda_1 \geq 0.474$ have been included in the results for SS-curve. For the EWM approaches, those cross-sections in which the reduced cross-sectional area needs to be considered have been included in the analysis.

Table 3. Assessment of design approaches for members in compression with slender cross-sections.

Grade		DSM CS-curve		DSM SS-curve		EWM approaches	
		$P_{DSM,EN}/P_u$	$P_{DSM,AS}/P_u$	$P_{DSM,EN}/P_u$	$P_{DSM,AS}/P_u$	P_{EN}/P_u	P_{AS}/P_u
Austenitic	Mean	0.98	0.87	0.85	0.74	0.91	0.82
	COV	0.101	0.089	0.110	0.089	0.119	0.132
Ferritic	Mean	0.93	0.88	0.83	0.79	0.85	0.81
	COV	0.080	0.088	0.092	0.095	0.093	0.179
Duplex	Mean	0.93	0.83	0.81	0.81	0.87	0.86
	COV	0.082	0.076	0.081	0.079	0.109	0.134
All	Mean	0.94	0.89	0.83	0.78	0.87	0.83
	COV	0.090	0.087	0.095	0.095	0.107	0.159

According to results presented in Table 3 and Figures 2 to 4, considerably different results are obtained for the DSM approach when carbon steel or stainless steel strength curves are considered, regardless of the adopted buckling curve. Although the stainless steel strength curve can be conservatively applied to the analysed SHS and RHS columns with slender cross-sections, more accurate results are obtained using the carbon steel strength curve given in Eq. (1). Comparing these results with the predicted capacities according to the codified EWM approaches, it is evident from Table 3 that the adoption of the DSM-based approach in conjunction with the carbon steel strength curve improves the flexural buckling resistance predictions for all the considered buckling curves and stainless steel grades in addition to obviating effective width calculations. Therefore, the DSM is a simpler and more accurate method for the prediction of the flexural buckling resistance of stainless steel SHS and RHS columns with slender cross-sections.

Regarding the most appropriate flexural buckling curves, Table 3 demonstrates that the highest predicted-to-experimental (or FE) ratios are obtained for the buckling curve codified in EN1993-1-4 [1] for both EWM and DSM approaches, although the flexural buckling capacity of some columns is over-predicted. Accurate results are also obtained for the buckling curves codified in AS/NZS4673 [2].

Nonetheless, it is evident from the column data showing high P_u/P_{ne} ratios in Figures 2 to 4 that some of the members with low cross-section slenderness exhibit an important strength reserve, mainly due to strain hardening effects. This fact is, as mentioned previously, more evident for austenitic stainless steels than ferritic and duplex grades. Therefore, a similar procedure to that presented in [11] is proposed in this paper to account for strain hardening effects in calculating the resistance of stainless steel SHS and RHS columns. The approach suggested in [9] for the incorporation of enhanced material properties in stainless steel columns, given in Eq. (6), provides for a level of strain hardening that only depends on the member slenderness of the column, without considering the behaviour of the cross-section. For a short column with a slender cross-section, the method would therefore provide an enhanced column capacity P_{enh_ne} that would subsequently be reduced according to the corresponding strength curve due to local buckling effects. The approach presented in this paper is slightly different and considers both member and cross-section slenderness when determining the level of strain hardening to be assigned to each column. Considering the conclusions extracted from Figures 2 to 4 and from [11] for cross-section behaviour, only results corresponding to the carbon steel strength curve will be considered. Therefore, the proposed full slenderness range expression for stainless steel columns is based on Eq. (5).

However, additional restrictions are required to prevent the unsafe prediction of the resistance of slender members due to the consideration of the enhanced material properties according to Eq. (5). Low cross-section slenderness λ_l values calculated from Eq. (2) may correspond to stocky cross-sections with a high local buckling critical load P_{crit} , and as such capable of reaching higher stresses than the 0.2% proof stress, (or yield stress). Hence, the adoption of a design approach

which considers strain hardening effects would introduce considerable improvements to the prediction of the capacity of these members. However, when low λ_1 values are caused by low flexural buckling resistances P_{ne} , members will fail due to overall buckling and the cross-section will only reach low stress levels. For these columns, strain hardening effects do not need to be considered, and the adoption of enhanced strengths would result in an overestimation of the actual capacity of the columns. Therefore, the effect of overall buckling needs to be considered by introducing an additional limitation, *viz.* stresses at which the column fails need to be higher than the yield stress to guarantee that the member is stable enough to develop partial yielding of the cross-sections. This condition is given in Eq. (17) and can be rewritten as in Eq. (18).

$$\sigma_{enh_nl} = \frac{P_{enh_nl}}{A} = \chi \sigma_{0.2} \left[1 + (1 - 1.29\lambda_1) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1 \right) \right] \geq \sigma_{0.2} \quad (17)$$

$$\chi \left[1 + (1 - 1.29\lambda_1) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1 \right) \right] \geq 1 \quad (18)$$

The assessment of the proposed modification to the DSM approach based on the carbon steel strength curve which considers the enhanced material properties (Eq. (5)) is presented in Table 4. In using Eq. (5), the column strength R_0 is calculated using the relevant buckling curve P_{ne} and then the cross-section slenderness λ_1 is calculated from Eq. (2). When the cross-section is assumed to be stocky ($\lambda_1 \leq 0.776$), and provided that Eq. (18) is satisfied, the enhanced column strength P_{enh_ne} is obtained from Eq. (5). Otherwise, if Eq. (18) is not satisfied, the column strength is taken as P_{ne} . Columns with a cross-section slenderness λ_1 lower than 0.776 have been considered in the evaluation of the DSM approach for stocky cross-sections and the strengths ($P_{DSM,EN}$, $P_{DSM,AS}$) are compared to those (P_{EN} , P_{AS}) obtained through the current EN1993-1-4 [1] and AS/NZS4673 [2] provisions. The mean values and COVs of the predicted-to-experimental (or FE) flexural buckling resistances of stainless steel columns with stocky cross-section are reported in Table 4 for the same buckling curves considered for slender cross-sections. Although the cross-sections are fully effective, results corresponding to the codified approaches have been denoted as EWM-based in Table 4. The predicted capacities of EWM-based approaches are limited to the yield stress, while DSM-based methods incorporate strain-hardening effects for those columns satisfying Eq. (18).

Table 4. Assessment of design approaches for members in compression with stocky cross-sections.

Grade		DSM CS-curve		EWM approaches	
		$P_{DSM,EN}/P_u$	$P_{DSM,AS}/P_u$	P_{EN}/P_u	P_{AS}/P_u
Austenitic	Mean	0.96	0.82	0.88	0.77
	COV	0.111	0.100	0.104	0.100
Ferritic	Mean	0.87	0.88	0.85	0.84
	COV	0.076	0.068	0.073	0.095
Duplex	Mean	0.88	0.91	0.85	0.84
	COV	0.050	0.057	0.052	0.102
All	Mean	0.88	0.87	0.85	0.83
	COV	0.085	0.074	0.077	0.100

Table 4 demonstrates that the DSM approach for columns with stocky cross-sections based on the carbon steel strength curve given in Eq. (5) provides improved results for all stainless steel grades and considered buckling curves, since higher P_{pred}/P_u ratios, with similar scatter, are obtained. However, as mentioned before, strain hardening effects cannot be included for all specimens with low cross-section slenderness. In order to evaluate the improvement introduced by the new proposal, only specimens with $\lambda_1 \leq 0.776$ that also fulfil the limitation given in Eq. (18) have been considered in the assessment presented in Table 5. Nevertheless, it should be noted that these columns do not represent a substantial portion of the analysed experimental and FE data, since both low member and cross-sectional slenderness are required for enhanced material properties to be considered.

Table 5. Assessment of design approaches for stocky members in compression with stocky cross-sections.

Grade		DSM CS-curve		EWM approaches	
		$P_{DSM,EN}/P_u$	$P_{DSM,AS}/P_u$	P_{EN}/P_u	P_{AS}/P_u
Austenitic	Mean	0.97	0.89	0.83	0.77
	COV	0.115	0.076	0.112	0.094

Ferritic	Mean	0.94	0.94	0.92	0.93
	COV	0.069	0.072	0.060	0.062
Duplex	Mean	0.90	0.91	0.87	0.89
	COV	0.036	0.035	0.041	0.041
All	Mean	0.94	0.93	0.90	0.89
	COV	0.081	0.072	0.080	0.097

According to the results presented in Table 5, which only considers stocky columns with stocky cross-section, the consideration of the enhanced material properties in the prediction of the ultimate capacity of stocky stainless steel columns based on Eq. (5) improves the obtained results for all buckling curves, although more significant improvements are obtained for austenitic stainless steel columns than for duplex and ferritic steel columns. According to Table 5, overall the best results are observed for the buckling curve codified in EN1993-1-4 [1] when enhanced material properties are considered, although for duplex grades the difference between the strengths ($P_{DSM,EN}$, $P_{DSM,AS}$) is marginal since the buckling curves corresponding to both standards are very similar.

4.3 RELIABILITY ANALYSIS

This section presents a reliability analysis of the proposed DSM approach for stainless steel SHS and RHS members subjected to compression. The calibration has been conducted following the procedure provided in Section F of the AISI-S100-12 Specification [5] where the statistical parameters corresponding to the material and geometrical variations of the different stainless steel grades have been extracted from Afshan et al. [39]. The considered material mean-to-nominal ratios are 1.3 for austenitic stainless steel, 1.2 for ferritics and 1.1 for duplex and lean duplex grades, with COVs equal to 0.060, 0.045 and 0.030 respectively. The mean-to-nominal ratio and COV of the geometric properties were taken as 1.0 and 0.050, respectively. The Australian and American codes prescribe a resistance factor ϕ_c equal to 0.9 for tubular cross-sections columns in compression and a target reliability index $\beta_0=2.5$, as adopted in [5] and [6] for structural members in LRFD. In reliability analyses, calculated reliability indices β need to be compared with the target value β_0 and approaches can be considered safe to apply for the resistance factor prescribe in the code when the calculated reliability index β is greater than the target value β_0 . In the

calculation of the different reliability indices, the load data and load factors from the Commentary of AS/NZS4600 [40] have been considered and a dead-to-live load ratio of 1/5 has been assumed. Table 6 presents the calculated reliability indices for the full slenderness range DSM approach for austenitic, ferritic and duplex stainless steel columns considering the buckling curves codified in EN1993-1-4 [1] and AS/NZS4673 [2]. Table 6 presents reliability indices for both stocky and slender cross-sections. The indices are fairly uniform for a given alloy, suggesting a fairly uniform reliability over the local buckling slenderness range.

Table 6. Summary of the reliability analysis results for the full slenderness range DSM for members in compression, $\phi_c=0.9$.

Grade	Calculated reliability indices β			
	Stocky cross-sections Enhanced mat. properties		Slender cross-sections Local buckling	
	EN1993-1-4	AS/NZS4673	EN1993-1-4	AS/NZS4673
Austenitic	2.98	3.54	3.11	3.64
Ferritic	3.13	3.10	3.16	3.40
Duplex	3.08	3.01	2.79	2.87

Considering the resistance factor ϕ_c equal to 0.9 as prescribed in AS/NZS4673 [1] and SEI/ASCE 8-02 [3] and the target reliability index of $\beta_0=2.5$, the results shown in Table 6 demonstrate that the proposed approach can be safely applied to members in compression for all the studied stainless steel grades according to the reliability requirements in [2] and [5], which have been adopted since the DSM is a design method closely associated with Australian and American codes. However, it is worth mentioning that the target reliability index established in EN1990 [41], $\beta_0=3.8$, is significantly higher than the value considered in this study. On this basis, and according to the results shown in Table 6, the proposed DSM approach would need of more in deep study to be safely included in EN1993-1-4 [1].

5. FULL SLENDERNESS RANGE DSM APPROACH FOR MEMBERS UNDER COMBINED LOADING

This section presents the assessment of the DSM-based method for beam-columns proposed by Rasmussen [10] also considered as a full slenderness range approach which includes both

enhanced material properties and local buckling effects. The method has been extended to austenitic, ferritic and duplex stainless steel SHS and RHS members subjected to compression and bending moment gradients, and the results are compared to strength predictions obtained from traditional interaction expressions in standards and the literature. The accuracy of the DSM approach is highly dependent on the accurate prediction of the flexural buckling P_{ne} and bending moment M_{ne} capacities, due to the adoption of Eq. (9) as reference equation, as well as on the considered interaction expression for the member behaviour k and the equivalent moment factor C_m featuring in the same equation. Based on the previous investigations on SHS and RHS beams reported in [11], the bending moment resistance calculated considering the inelastic capacity reserve with the compression strain factor C_y provided in Eq. (8) has been considered in the beam-column analysis.

5.1 INTERACTION APPROACHES FOR STAINLESS STEEL BEAM-COLUMNS

Different design expressions can be found in standards and the literature for the evaluation of stainless steel members subjected to combined loading. Nevertheless, beam-column behaviour verifications are usually presented as interaction expressions with the same general expression, given by Eq. (9), and a certain interaction factor k . The differences among available interaction equations are found in the definition of the interaction factor k and the calculation of the basic flexural buckling P_{ne} and bending moment M_{ne} capacities. The interaction factor codified in EN1993-1-4 [1] is defined by Eq. (19), where the minimum value of 1.2 is worth mentioning, since it usually produces overly conservative capacity predictions as the full bending capacity of the cross-section M_{ne} cannot be reached for low compression values. Other standards for stainless steel, such as SEI/ASCE 8-02 [3] and AS/NZS4673 [2], consider an interaction factor k with an amplification factor which depends on the critical elastic column load for flexural buckling P_{cre} , given by Eq. (20). Alternatively, Method 2 in EN1993-1-1 [14] for carbon steel provides the interaction factor in Eq. (21), which has also been included in the present analysis for comparison.

$$k = 1 + 2(\lambda_c - 0.5) \frac{P_{one}}{P_{ne}} \quad \text{but} \quad 1.2 \leq k \leq 1.2 + 2 \frac{P_{one}}{P_{ne}} \quad (19)$$

$$k = \frac{1}{1 - \frac{P_{one}}{P_{cre}}} \quad (20)$$

$$k = 1 + 0.6\lambda_c \frac{P_{one}}{P_{ne}} \leq 1 + 0.6 \frac{P_{one}}{P_{ne}} \quad (21)$$

Recent research on experimental and FE stocky stainless steel SHS and RHS beam-columns subjected to different bending moment distributions was carried out by Zhao et al. [35]. A new expression for the interaction factor k which also considers the different behaviour of diverse stainless steel grades was proposed based on the interaction factor suggested by Greiner and Kettler [42]. The proposed interaction factor is given in Eq. (22) and the calibrated D_1 , D_2 and D_3 parameter values for different stainless steel grades can be found in the original publication. This proposal was based on an alternative flexural buckling resistance approach given by Afshan et al. [43] and the pure bending moment resistance determined according to the Continuous Strength Method (CSM), where the effect of strain hardening is considered. This flexural buckling approach has not been included in the previous stainless steel column investigations since the only available information for the new buckling curves is the short summary given in [35] but is used herein in the assessment of the interaction expression.

$$k = 1 + D_1(\lambda_c - D_2) \frac{P_{one}}{P_{ne}} \leq 1 + D_1(D_3 - D_2) \frac{P_{one}}{P_{ne}} \quad (22)$$

5.2 BEAM-COLUMNS UNDER UNIFORM BENDING MOMENT

The DSM-based approach for beam-columns developed by Rasmussen [10] is analysed for austenitic, ferritic and duplex stainless steel SHS and RHS members subjected to compression and uniform bending using the different flexural buckling resistances P_{ne} and interaction expressions k introduced in previous sections. Since the different interaction factors include amplification of moments, they implicitly assume the moments are obtained from a first order (linear) analysis. Accordingly, for beam-columns subjected to constant eccentricity of loading e , the moment and equivalent moment factor to be substituted into the interaction equations presented in Section 2.3 are to be taken as $M_{one}=P_{one} \cdot e$ and $C_m=1$, respectively. Although the original proposal only included the capacity reduction due to local buckling effects in slender

cross-sections, the approach has been investigated as a full cross-sectional slenderness which also considers the enhanced material properties. Results obtained using the DSM design approach have been compared to the predicted beam-column strengths obtained from the codified approaches through the traditional effective width method to identify the most accurate approach. The assessment of the DSM approach for beam-columns under uniform bending moment distribution is presented in Figures 5 to 7 for austenitic, ferritic and duplex stainless steel grades, respectively. Results corresponding to the interaction expressions codified in EN1993-1-4 [1] and AS/NZS4673 [2], with interaction factors k defined in Eqs. (19) and (20), are presented in the (a)-figures for each material, followed by the (b)-figures containing the EN1993-1-1 [14] and Zhao et al. [35] approaches with interaction factors k introduced in Eqs. (21) and (22). In the figures, the r_u/r_{ne} ratio is plotted against the corresponding generalized slenderness λ_n calculated from Eq. (12), where local elastic critical buckling loads were obtained from CUFSM calculations. For comparison, Figures 5 to 7 also show carbon steel and stainless steel strength curves as defined in Eqs. (1) and (3). Figures 5 to 7 suggest that the strength curve given in Eq. (1) for carbon steel cross-sections provides better results for the analysed beam-columns, showing a similar agreement to that exhibited by cross-sections and columns in Section 4.2. The scatter observed in the low slenderness range for all materials and investigated interaction factors can be partly attributed to strain hardening effects when stocky members and cross-sections are considered, and partly to the inaccuracy of the analytic expressions used in the prediction of the flexural buckling and overall-local buckling interaction behaviour of the specimens. These figures show all the available stocky and slender beam-column data, without including the limitation to account for enhanced material properties equivalent to that proposed for stainless steel columns, which will be introduced hereafter.

a) AS/NZS4673 and EN1993-1-4 approaches

b) Zhao et al. [35] and EN1993-1-1 approaches

Figure 5. Assessment of the DSM approach for austenitic stainless steel SHS and RHS beam-columns under uniform bending for different interaction and buckling curves.

a) AS/NZS4673 and EN1993-1-4 approaches

b) Zhao et al. [35] and EN1993-1-1 approaches

Figure 6. Assessment of the DSM approach for ferritic stainless steel SHS and RHS beam-columns under uniform bending for different interaction and buckling curves.

a) AS/NZS4673 and EN1993-1-4 approaches

b) Zhao et al. [35] and EN1993-1-1 approaches

Figure 7. Assessment of the DSM approach for duplex stainless steel SHS and RHS beam-columns under uniform bending for different interaction and buckling curves.

The assessment of the full slenderness range DSM approach has been derived analysing slender and stocky cross-sections separately in order to evaluate both the overall-local buckling

interaction and strain hardening effects. The numerical assessment of the new approach for beam-columns under uniform bending distribution is presented in Tables 7 and 8 for slender and stocky cross-sections respectively. Based on the conclusions extracted in [11] for cross-sections, from the previous analysis on columns and the observations made from Figures 5 to 7, only results corresponding to the strength curve provided for carbon steel members are considered and reported in the following analyses. These results have also been compared with the results obtained for the same interaction expressions but considering the local buckling effects through the Effective Width Method (EWM) reported in the Tables, where no strain-hardening effects are introduced except for the Zhao et al. [35] approach, since the latter was based on the cross-sectional bending moment resistance calculated according to the CSM and these effects are therefore implicit in Eq. (22).

Results reported in Table 7 correspond to stainless steel beam-columns with slender cross-sections having generalized slenderness higher than $\lambda_n \geq 0.776$ subjected to compression and uniform bending moment distribution. According to the results, the DSM approach improves the capacity prediction of stainless steel beam-columns for all material grades and considered interaction expressions, since the average r_{pred}/r_u ratios corresponding to the DSM are closer to unity than those in obtained for the EWM, and the COVs are lower. This improvement is caused by two factors. First, considering the actual stress distribution in the cross-section when determining the generalized slenderness λ_n provides a better estimation of the actual susceptibility of the cross-section to local buckling effects; and second, the strength curve adopted for the member capacity reduction due to local buckling is more accurate than the reduction factors ρ for the EWM. In addition, the most accurate DSM capacity predictions are obtained for the EN1993-1-1 [14] interaction curve given in Eq. (21) with the flexural buckling resistance determined according to EN1993-1-4 [1] and the bending resistance calculated considering the inelastic capacity reserve as reported in [11].

Table 7. Assessment of the DSM design approach for beam-columns with slender cross-section and uniform bending distribution for different interaction expressions.

Grade		DSM CS-curve				EWM approaches			
		r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u	r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u
Austenitic	Mean	0.90	0.87	0.94	0.89	0.81	0.76	0.85	0.81
	COV	0.073	0.070	0.065	0.074	0.086	0.100	0.091	0.081
Ferritic	Mean	0.86	0.85	0.89	0.87	0.78	0.78	0.83	0.80
	COV	0.066	0.072	0.062	0.089	0.093	0.106	0.082	0.125
Duplex	Mean	0.92	0.92	0.96	0.91	0.83	0.82	0.86	0.83
	COV	0.087	0.077	0.078	0.077	0.106	0.106	0.091	0.106
All	Mean	0.88	0.87	0.92	0.88	0.80	0.78	0.84	0.81
	COV	0.078	0.080	0.074	0.086	0.098	0.109	0.087	0.115

Beam-columns with stocky cross-section can attain stress levels beyond the yield stress if the member is sufficiently stable to develop partial yielding of the cross-section. As evidenced in the previous section for the consideration of strain hardening effects in columns, low generalized slenderness λ_n values might correspond to stocky cross-sections with the corresponding high r_{cr1} parameters, but also to slender member with low r_{ne} parameters. Hence, the effect of overall member behaviour needs to be considered by introducing additional limitations. The definition of this limitation is not as clear as for columns, where the stress at which the column fails can be limited to the 0.2% proof stress, since compression loads interact with bending moments. However, following a similar approach to that for columns, the condition presented in Eq. (23) is proposed for beam-columns. This condition establishes that the enhanced member capacity should always be higher than the yield capacity of the cross-section, given by the r_y parameter. This r_y factor corresponds to the cross-section yield strength extensively investigated by Arrayago et al. [11] for combined loading conditions. Considering the definitions of r_{ne} and r_y given in Section 2.3 and [11], the condition for considering strain hardening effects leads to Eq. (24) and as for uniform bending distributions C_m is equal to unity, this condition can be re-written as in Eq. (25).

$$r_{enh_nl} = r_{ne} \left[1 + (1 - 1.29\lambda_n) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1 \right) \right] \geq r_y \quad \text{for } \lambda_n \leq 0.776 \quad (23)$$

$$\frac{P_{one}}{P_y} \left[1 + (1 - 1.29\lambda_n) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1 \right) \right] \frac{\sqrt{\left(\frac{1}{P_y} \right)^2 + \left(\frac{eC_m}{M_y} \right)^2}}{\sqrt{\left(\frac{1}{P_y} \right)^2 + \left(\frac{e}{M_y} \right)^2}} \geq 1 \quad (24)$$

$$\frac{P_{one}}{P_y} \left[1 + (1 - 1.29\lambda_n) \left(\frac{\sigma_u}{\sigma_{0.2}} - 1 \right) \right] \geq 1 \quad (25)$$

The assessment of the DSM approach for beam-columns with stocky cross-sections, with generalized slenderness lower than $\lambda_n \leq 0.776$ and uniform bending moment diagram, is presented in Table 8 for the different interaction expressions. For those specimens satisfying the condition given in Eq. (25), the enhanced material properties have been considered using Eq. (5). Table 8 also shows the r_{pred}/r_u ratios corresponding to the EWM for the same interaction expressions. The results demonstrate that overall, DSM predictions are more accurate for all stainless steel grades and all interaction expressions, obtaining excellent results for the EN1993-1-1 [14] interaction curve which are consistent with those shown in Table 7 for beam-columns with slender cross-sections. As highlighted for stainless steel columns in the previous section, the improvement introduced in the beam-column capacity prediction is more evident for austenitic and duplex stainless steel grades. For ferritic alloys, the differences are smaller since these specimens are characterized by low $\sigma_u/\sigma_{0.2}$ ratios.

Table 8. Assessment of the DSM design approach for beam-columns with stocky cross-section and uniform bending distribution for different interaction expressions.

Grade		DSM CS-curve				EWM approaches			
		r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u	r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u
Austenitic	Mean	0.89	0.87	0.98	0.88	0.84	0.82	0.91	0.91
	COV	0.138	0.108	0.107	0.110	0.097	0.070	0.093	0.095
Ferritic	Mean	0.82	0.85	0.91	0.88	0.83	0.85	0.90	0.85
	COV	0.089	0.060	0.065	0.080	0.088	0.061	0.082	0.084
Duplex	Mean	0.85	0.87	0.92	0.86	0.81	0.86	0.86	0.87
	COV	0.063	0.050	0.052	0.044	0.072	0.036	0.059	0.048
All	Mean	0.84	0.86	0.93	0.85	0.83	0.84	0.90	0.87
	COV	0.106	0.073	0.083	0.083	0.089	0.061	0.084	0.087

5.3 BEAM-COLUMNS UNDER BENDING MOMENT GRADIENT

With the DSM approach assessed for stainless steel members subjected to compression and uniform bending, the method has been extended to members with non-uniform bending moment

diagrams. The effect of the bending moment gradient is accounted for by including an equivalent moment factor C_m in the general interaction equation presented in Eq. (9). EN1993-1-4 [1] provisions do not account for the effect of the bending moment gradient in the member behaviour and the same interaction expression is provided for uniform and non-uniform bending moment distributions. However, the different design standards for carbon steel and stainless steel beam-columns (i.e. EN1993-1-1 [14], AS/NZS 4600 [40], AISI-S100-12 [5], AS/NZS4673 [2] and SEI/ASCE 8-02 [3]) adopt the equivalent uniform moment factor given in Eq. (26), in which ψ is the ratio of the smaller to the larger moment at the ends, already defined in Section 3.2. The full slenderness range DSM approach has been extended to non-uniform bending moment distributions by assuming a small modification of the procedure described in Eqs. (9) to (13), where an equivalent load eccentricity as given by $e_{eq}=C_m \cdot e$ is adopted in order to obtain accurate and safe capacity predictions.

$$C_m = 0.6 + 0.4\psi \geq 0.4 \quad (26)$$

In the design of beam-columns, both the cross-sectional and member resistance conditions need to be verified. Although for beam-columns subjected to uniform bending moment distributions the member verification is governing, for non-uniform bending diagrams the failure mechanism is not obvious and both failure conditions need to be checked. In the assessment of the DSM method, the cross-section capacity of the considered specimens has been determined according to the full slenderness range DSM approach proposed in Arrayago et al. [11] for cross-sections subjected to combined loading. Therefore, the following analysis corresponds only to those specimens showing overall or combined overall-local buckling failure modes.

Figures 8, 9 and 10 show the assessment of the DSM approach for beam-columns under non-uniform bending moment distributions assuming proposed modification for the equivalent load eccentricity e_{eq} for austenitic, ferritic and duplex stainless steel grades, respectively. As for Figures 5 to 7, results corresponding to the interaction expressions given in EN1993-1-4 [1] and AS/NZS4673 [2] are presented in the (a)-figures for each material, followed by the (b)-figures containing the EN1993-1-1 [14] and Zhao et al. [35] approaches. r_u/r_{ne} ratios have been plotted

against the corresponding generalized slenderness λ_n and DSM strength curves for carbon steel and stainless steel are also shown for comparison. These figures highlight that the strength curve given for carbon steel cross-sections provides better results also for beam-columns with non-uniform bending distributions, with higher scatter than for the cases presented before.

a) AS/NZS4673 and EN1993-1-4 approaches

b) Zhao et al. [35] and EN1993-1-1 approaches

Figure 8. Assessment of the DSM approach for austenitic stainless steel SHS and RHS beam-columns under non-uniform bending for different interaction and buckling curves.

a) AS/NZS4673 and EN1993-1-4 approaches

b) Zhao et al. [35] and EN1993-1-1 approaches

Figure 9. Assessment of the DSM approach for ferritic stainless steel SHS and RHS beam-columns under non-uniform bending for different interaction and buckling curves.

a) AS/NZS4673 and EN1993-1-4 approaches

b) Zhao et al. [35] and EN1993-1-1 approaches

Figure 10. Assessment of the DSM approach for duplex stainless steel SHS and RHS beam-columns under non-uniform bending for different interaction and buckling curves.

The assessment of the full slenderness range DSM approach for beam-columns subjected to non-uniform bending diagrams is also presented in Tables 9 and 10 for slender and stocky cross-sections respectively. The same interaction expressions assessed for the uniform bending distribution have been considered in the analysis, adopting the strength curve for carbon steel

cross-sections given in Eq. (5). The numerical assessment of the DSM approach for stainless steel beam-columns with slender cross-sections is presented in Table 9 where the mean and COVs of the predicted-to-experimental (or FE) ratios are reported. The results demonstrate that the DSM approach improves the strength prediction of stainless steel beam-columns with slender cross-sections when compared with the corresponding values for the EWM also shown in Table 9. As for uniform bending moment, this is due to the facts that (i) the actual stress distribution is considered when the generalized slenderness is calculated and (ii) a more accurate strength curve is adopted for local buckling. In addition, as for uniform bending, the best capacity prediction is again obtained for the EN1993-1-1 [14] interaction curve with the flexural buckling curve as codified in EN1993-1-4 [1].

Table 9. Assessment of the DSM design approach for beam-columns with slender cross-section and non-uniform bending distribution.

Grade		DSM CS-curve				EWM approaches			
		r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u	r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u
Austenitic	Mean	0.87	0.81	0.91	0.84	0.81	0.71	0.84	0.78
	COV	0.081	0.104	0.077	0.089	0.073	0.100	0.068	0.069
Ferritic	Mean	0.80	0.80	0.83	0.80	0.74	0.73	0.78	0.73
	COV	0.093	0.084	0.075	0.097	0.099	0.108	0.082	0.109
Duplex	Mean	0.88	0.88	0.92	0.88	0.79	0.79	0.83	0.79
	COV	0.096	0.098	0.086	0.079	0.087	0.108	0.086	0.090
All	Mean	0.82	0.82	0.86	0.82	0.76	0.74	0.79	0.75
	COV	0.101	0.094	0.091	0.100	0.100	0.112	0.087	0.107

For beam-columns with stocky cross-sections showing a generalized slenderness lower than $\lambda_n \leq 0.776$, the assessment of the DSM approach for the different interaction expressions is presented in Table 10. The results correspond to beam-columns subjected to non-uniform bending diagrams, where enhanced material properties have been considered when Eq. (24) is satisfied. When the results are compared with the values reported for the same interaction expressions based on the EWM, as also shown in Table 10, it can be appreciated that the DSM approach improves the capacity predictions for all stainless steel grades as for slender cross-sections with non-uniform bending moment distributions. Table 10 also demonstrates that the most accurate results are obtained for the EN1993-1-1 [14] interaction curve, although the capacity of a few austenitic specimens is overestimated using the equivalent uniform moment factor. The effect of these

overestimated strengths on the mean and COV of the prediction is considered in the reliability analysis presented in the following section.

Table 10. Assessment of the DSM design approach for beam-columns with stocky cross-section and non-uniform bending distribution.

Grade		DSM CS-curve				EWM approaches			
		r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u	r_{EN1-4}/r_u	$r_{AS/NZS}/r_u$	r_{EN1-1}/r_u	r_{Zhao}/r_u
Austenitic	Mean	0.94	0.88	1.02	0.92	0.87	0.78	0.90	0.87
	COV	0.169	0.146	0.154	0.143	0.078	0.061	0.077	0.090
Ferritic	Mean	0.79	0.82	0.87	0.77	0.80	0.78	0.83	0.77
	COV	0.129	0.090	0.087	0.126	0.105	0.073	0.090	0.095
Duplex	Mean	0.84	0.86	0.90	0.86	0.82	0.81	0.86	0.83
	COV	0.127	0.120	0.127	0.116	0.079	0.089	0.065	0.069
All	Mean	0.83	0.83	0.90	0.81	0.82	0.79	0.86	0.80
	COV	0.155	0.114	0.132	0.150	0.102	0.076	0.090	0.102

5.4 RELIABILITY ANALYSIS

The reliability of the proposed full slenderness range DSM approach for SHS and RHS beam-columns is assessed using the same reliability analysis as that described in Section 4.3. The assessment is again based on a resistance factor ϕ_c equal to 0.9 and assumes a target reliability index of $\beta_0=2.5$. Table 11 presents the calculated reliability indices for the full slenderness range DSM approach for austenitic, ferritic and duplex stainless steel beam-columns subjected to uniform bending distributions, while Table 12 reports the indices corresponding to non-uniform distributions. Considering the resistance factors ϕ_c prescribed in AS/NZS4673 [2] and SEI/ASCE 8-02 [3], the results shown in Tables 11 and 12 demonstrate that the proposed approach for beam-columns can be safely applied to all the studied stainless steel grades and bending distributions.

Table 11. Summary of the reliability analysis results for the full slenderness range DSM for beam-columns with uniform bending distribution, $\phi_c=0.9$.

Grade	Calculated reliability indices β			
	Stocky cross-sections Enhanced mat. properties		Slender cross-sections Local buckling	
	AS/NZS4673	EN1993-1-1 and EN1993-1-4	AS/NZS4673	EN1993-1-1 and EN1993-1-4
Austenitic	3.62	3.07	3.77	3.47
Ferritic	3.62	3.30	3.47	3.33
Duplex	3.14	2.91	2.84	2.59

Table 12. Summary of the reliability analysis results for the full slenderness range DSM for beam-columns with non-uniform bending distribution, $\phi_c=0.9$.

Grade	Calculated reliability indices β			
	Stocky cross-sections Enhanced mat. properties		Slender cross-sections Local buckling	
	AS/NZS4673	EN1993-1-1 and EN1993-1-4	AS/NZS4673	EN1993-1-1 and EN1993-1-4
Austenitic	3.26	2.66	3.83	3.48
Ferritic	3.67	3.43	3.76	3.67
Duplex	2.85	2.69	2.95	2.79

6. CONCLUSIONS

This paper proposes a full slenderness range Direct Strength Method (DSM) approach for stainless steel SHS and RHS members subjected to compression and combined compression and bending. The proposal is based on the DSM strength curve provided for carbon steel cross-sections in the AISI-S100-12 specification [5] and the method for incorporating strain hardening proposed in [11]. The new approach was found to be more accurate for both columns and beam-columns than the provisions of current specifications for stainless steel structural members. This applied to both stocky and slender cross-sections, partly because strain hardening effects are incorporated and partly because the actual stress distribution of the cross-section is considered when determining the elastic buckling stress. This provides a better estimation of the susceptibility of the cross-section to local buckling effects. The adopted strength curve for the member capacity reduction due to local buckling was also found to be more accurate than the reduction factors codified in standards. Although an additional limitation needs to be imposed before strain hardening effects can be included to increase the member strength, it has been demonstrated that the same strength curve proposed for determining cross-sectional strength can also be applied to stainless steel members. The reliability of the proposed DSM approaches for columns and beam-columns was assessed following the procedures provided in the AISI-S100-12 specification [5] and it was demonstrated that the DSM strength equations can be safely applied.

It was also demonstrated that for SHS and RHS stainless steel columns, the most accurate capacity predictions are obtained when the buckling curve codified in EN1993-1-4 [1] is considered for

the determination of the basic flexural buckling resistance P_{ne} for all the studied stainless steel grades. Finally, it has been demonstrated that the most accurate capacity predictions for stainless steel beam-columns are obtained by using the EN1993-1-1 [14] interaction curve, adopting the flexural buckling resistance determined according to EN1993-1-4 [1] and calculating the bending resistance considering the inelastic capacity reserve as reported in [11]. The method has been extended to non-uniform bending moment distributions by assuming an equivalent load eccentricity and the equivalent uniform moment factor C_m currently included in EN1993-1-1 [14], AS/NZS4673 [1] and SEI/ASCE 8-02 [3].

Further research needs to be conducted on open sections, such as I-sections and channel sections, in order to identify the strength curve for open sections which best predicts the overall-local buckling interaction for the loading cases analysed in this paper. The proposed approach for incorporating strain hardening effects should also be assessed for these sections.

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