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SECTIONAL BUCKLING DESIGN OF BUILT-UP COLD-FORMED STEEL COLUMNS

Mandana Abbasi*, Kim J.R. Rasmussen*, Mani Khezri and Benjamin W. Schafer**

* School of Civil Engineering, The University of Sydney, Sydney, Australia
e-mails: mandana.abbasi@sydney.edu.au, kim.rasmussen@sydney.edu.au, mani.khezri@sydney.edu.au

** Department of Civil Engineering, Johns Hopkins University, Baltimore, USA
e-mail: schafer@jhu.edu

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Abstract. *The paper investigates the applicability of the current design methods for local and distortional (sectional) buckling of built-up cold-formed steel columns through comparisons with the results of numerical parametric studies. A detailed reliability analysis is performed to assess the suitability of the current design methods for the sectional buckling of built-up cold-formed steel members in compression. The results suggest that the Direct Strength Method, combined with the Compound Strip Method for the elastic buckling analysis compared to the classical Finite Strip Method, performs better in predicting the distortional buckling strength while still providing safe and efficient predictions of the local buckling strength. In addition, adjustments are proposed to improve the accuracy of the current direct strength design equations for the ultimate capacity predictions of built-up compression members to comply with the required level of reliability in the Australian and North American provisions while conserving the prescribed strength reduction factor. The proposed modifications are shown to be reasonably accurate and robust for different cross-section and fastener configurations.*

1 INTRODUCTION

Various methods exist in the literature for the design of cold-formed steel (CFS) members, of which the Effective Width Method (EWM) and the Direct Strength Method (DSM) are those most common used. The EWM is a semi-empirical method for modelling the stress distribution in the post-buckling regime of thin plates, which was first introduced by von Karman et al. [1] and later improved by Winter [2]. EWM is among the first methods for designing thin-walled members and has been employed in many design codes [3-5]. The underlying idea of the EWM for CFS sections lies in finding the effective width for each plate element and then reassembling the effective constituents to evaluate the effective overall section capacity. This method generally neglects the probable interactions between plate elements at buckling or throughout the post-buckling regime [6], which may not necessarily be accurate for built-up sections. Limited studies have been conducted on the applicability and accuracy of the conventional EWM for built-up columns, e.g. [7, 8].

The basic idea and concept of the DSM were initially proposed by Hancock [9], in which a correlation was suggested between the ultimate strength and the elastic buckling stress of CFS columns under distortional buckling. Later, this alternative design approach was extended [10, 11] for a broader range of cross-sections failing in local, distortional, flexural, or flexural-torsional buckling modes. As a result, the DSM has been successfully incorporated in international standards, including the Australian standard [3] and North American specification [4] for the design of CFS members. Despite being extensively validated for single CFS sections, its accuracy and calibration for the design of built-up sections remains an active area of research and efficient, generally applicable design provisions for the sectional buckling strength are still to be proposed.

In a study on built-up CFS closed sections with intermediate stiffeners by Young and Chen [12], the obtained test strengths were compared with design capacities determined using the DSM. Three sections were analysed: the single section, the single section restrained at the flanges and the double section with the flange thickness twice the web thickness. It was concluded that assuming no interaction between component profiles results in acceptable but conservative capacity predictions in built-up closed sections with stiffeners. Later, Zhang and Young [13, 14] investigated the appropriateness of the DSM for built-up CFS I-shaped and box members, where the test data were compared with the DSM capacity of composite sections with assumed web thicknesses ranging from 1.0 to 2.0 times the thickness of a single section. It was reported that assuming full composite action can overestimate the expected ultimate strength of stiffened built-up open sections and an adjustment method based on introducing a scaling factor for the web thickness was suggested. This finding indicates that a degree of composite action between component sections is achieved in built-up members due to the presence of fasteners, which must be carefully considered in determining the elastic buckling stresses to extend the DSM to the design of built-up sections [15].

Zhang and Young [13] proposed a method to model screws as a continuous solid stiffener along the longitudinal direction of the column. They presented a modified DSM and showed that predicted design strengths are reliable and conservative for built-up open-section columns. Georgieva et al. [16] investigated the applicability of the DSM for the design of built-up CFS through experimental studies and suggested that a similar modified design methodology based on DSM can be adopted in structural standards. A modified DSM for the cold-formed built-up I-section columns was presented by Lu et al. [17], who showed that the current DSM can lead to excessively unsafe estimates for built-up I-section columns. Moreover, Phan et al. [18] proposed a new design approach for built-up columns with complex geometries using the current direct strength equations and the effective rigidities derived theoretically in [19, 20].

In summary, the focus of the previous work for extending the use of current design equations to built-up CFS columns has mainly been on either built-up sections comprised of two component sections or proposing a simplified method for calculating the critical buckling stresses of the built-up sections. Therefore, this study aims to assess the reliability of current design procedures based on the numerical parametric studies for built-up sections with more than two component sections and to demonstrate the versatility of the Compound Strip Method (CSM) [21] to provide accurate solutions for the elastic buckling analysis of built-up members. Adjustments are subsequently proposed to enhance the accuracy of direct design equations in predicting the sectional strength of built-up columns to meet the target reliability index in the Australian and North American specifications.

2 CURRENT DESIGN METHODS FOR COLD-FORMED STEEL MEMBERS

The current design methods in Australian [3], North American [4] and European [5] standards include the conventional EWM, which modifies the width of the constituent plate elements for sectional capacity calculations based on their buckling state. Although the effective width approach in the Australian standard AS4600 [3] and North American specification AISI S100 [4] shares the same principles as in the European standard [5], it has some differences with some caveats and additional rules for built-up members. For instance, for built-up compression members made of two sections in contact, such as a back-to-back connection of two channel sections forming an I-section, the global slenderness ratio (l_e/r_g) can be modified due to the potential enhancement of the buckling capacity as long as the buckling mode involves relative deformations of the sections imposing shear demand on the fasteners.

An alternative design procedure is the DSM, which correlates the ultimate capacity of the member with the critical buckling loads of the section. The nominal axial capacity (P_n) of a CFS compression member in DSM is defined as the minimum of the nominal axial strengths for the member (flexural, torsional or flexural-torsional) buckling (P_{ne}), local buckling (P_{nl}) and distortional buckling (P_{nd}) limit states. The direct strength design equations for different buckling modes can be written in the following generalised form,

$$\frac{P_{ni}}{P_{mi}} = \begin{cases} 1 & \forall \lambda_i \le \lambda_{i.\lim} \\ \left(a_i - \frac{b_i}{\lambda_i^{c_i}}\right)\frac{1}{\lambda_i^{c_i}} & \forall \lambda_i > \lambda_{i.\lim} \end{cases} \tag{1}$$

where $P_{mi}=f_{mi}A_g$ is the maximum nominal capacity associated with the mode of interest (i) acting as the cap for the ultimate capacity, and λ_i is the slenderness ratio for the buckling mode i that is proportional to the ratio of peak strength (f_{mi}) to the critical buckling stress (f_{cri}) in that mode, i.e. $\lambda_i=\sqrt{(f_{mi}/f_{cri})}$. Furthermore, a_i, b_i and c_i are the coefficients corresponding to the mode of interest (i) that are typically obtained from regression to test data to meet an acceptable level of target reliability index in design. Lastly, $\lambda_{i.lim}$ is the limiting slenderness ratio associated with each buckling mode that must adhere to the following correlation for compatibility between the conditions:

$$\lambda_{i.\lim} = \left[\frac{a_i - \sqrt{a_i^2 - 4b_i}}{2b_i}\right]^{\frac{-1}{c_i}} \tag{2}$$

The key parameters of the current direct strength equations for local and distortional buckling are summarised in Table 1.

Table 1: Parameters of current direct strength design equations for sectional buckling

Buckling Mode	a_i	b_i	c_i	$\lambda_{i.\lim}$	P_{mi}
Local	1.0	0.15	0.8	0.776	P_{ne}
Distortional	1.0	0.25	1.2	0.561	P_y

3 ASSESSMENT OF THE CURRENT DESIGN RULES FOR BUILT-UP COLUMNS

3.1 Comparison with the results of parametric studies

Extensive parametric studies were carried out in a separate study [22] on various design scenarios, including the section depth, width, thickness, built-up section geometry and screw spacing to improve the understanding of the influencing parameters on the sectional buckling of built-up columns with enhanced coverage of slenderness ratios. Three values of section depth, i.e. h=100, 200, and 300 mm, with two different nominal thicknesses and two sets of flange width to web depth (b_f/h) ratios, were selected for each section depth such that the prevalent buckling mode of the section was the local buckling mode in set-L and the distortional mode in set-D. Figure 1 shows the geometry of the selected built-up sections for each set of parametric studies. The impact of intermediate discrete fasteners on the degree of coupling and interaction between individual sections was studied by considering six different screw spacing to length (s/L) ratios ranging from 1 to 1/16.

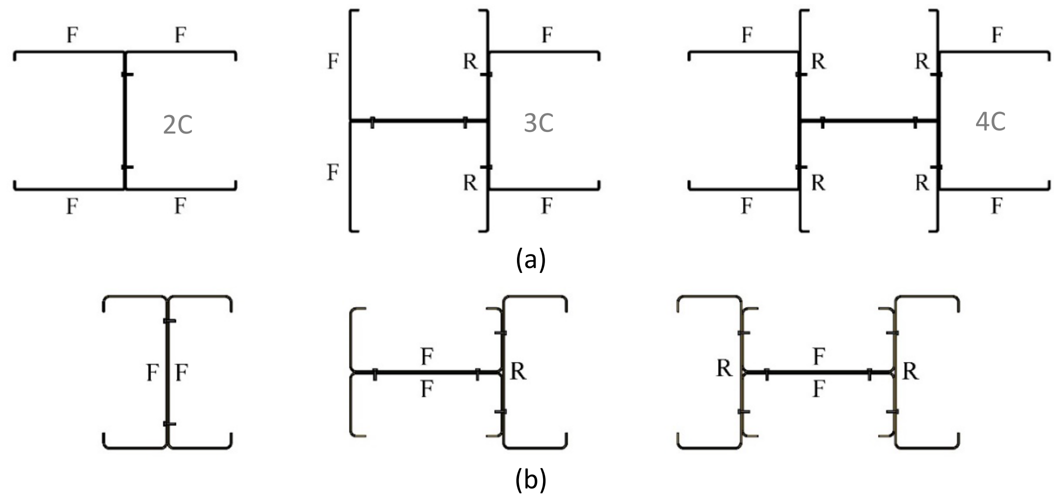


Figure 1: Built-up sections geometry used in parametric studies: (a) set-D, (b) set-L

The predictions of the current design strength equations are compared with the numerical FE data points obtained from the parametric studies. The comparisons are made separately for distortional and local buckling, and the results are provided in Figure 2 and Figure 3, respectively. It is noted that the numerical strengths are plotted in these figures with respect to two sets of slenderness ratios: 1) using the slenderness ratio corresponding to a single section (λ_1) without any coupling consideration between sections (referred to as NC and shown in the a-figures) and 2) using the slenderness of the built-up section (partially-composite assumption, denoted as PC and shown in the b-figures) obtained using the CSM [21].

The results suggest that the latter approach is more representative due to its capability to distinguish between the various fastener spacings considered and various levels of composite action attained, which produce a distributed scatter of data points in closer agreement with the direct strength curve (Figures 2b and 3b), rather than vertically distributed data points at discrete slenderness ratios, as shown Figures 2a and 3a. The DSM is generally conservative for the local buckling of built-up sections (Figure 3), especially in the intermediate slenderness ratios ($1.0 \leq \lambda_l \leq 2.0$). For distortional buckling (Figure 2), DSM reasonably captures the ultimate capacity of single sections and is reasonably accurate for built-up I-sections at moderate to high slenderness ratios. Nevertheless, it becomes conservative for more complex sections (3C, 4C) with mixed connectivity and members with relatively small fastener spacing ratios ($s/L_{crd} \leq 0.5$).

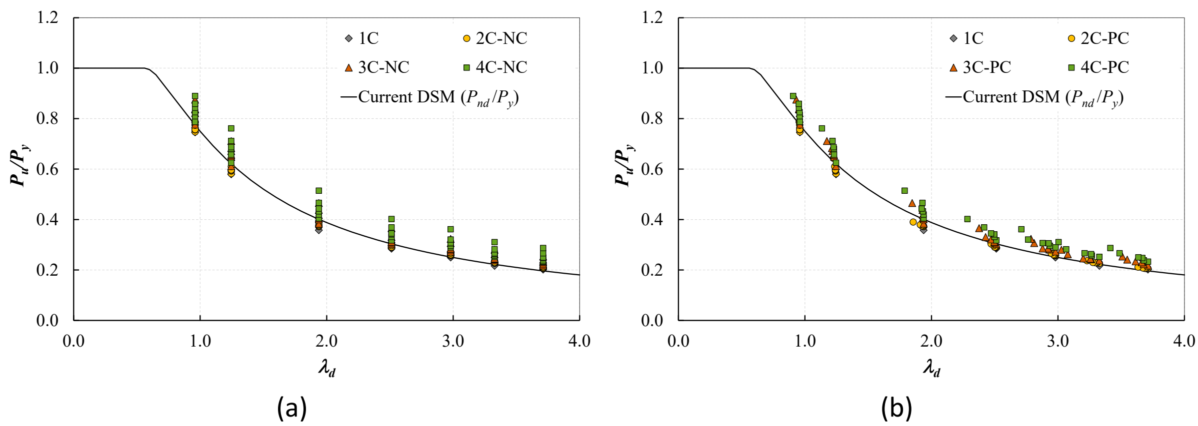


Figure 2: Variation of P_u/P_y with distortional slenderness ratio λ_d obtained with different levels of composite action: (a) non-composite, (b) partially-composite

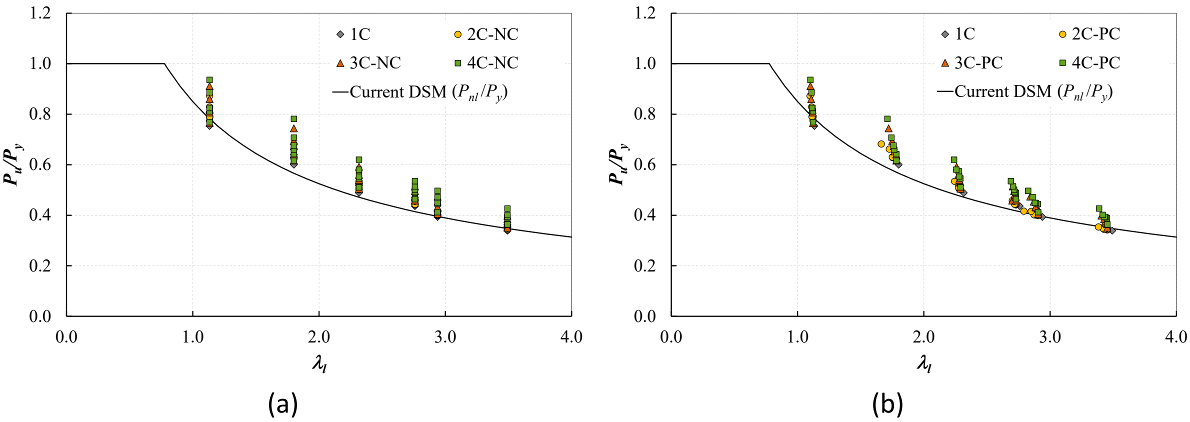


Figure 3: Variation of P_u/P_y with local slenderness ratio λ_l obtained with different levels of composite action: (a) non-composite, (b) partially-composite

3.2 Reliability analysis

3.2.1 Basic principles

The reliability index (β), as a commonly considered measure for the structural safety margin, can be evaluated from the following expression based on the First Order Second Moment (FOSM) Method [23, 24]:

$$\beta = \frac{\ln(P_m M_m F_m \, c_\phi / \phi)}{\sqrt{V_R^2 + V_S^2}} \tag{3}$$

in which ϕ is the strength reduction factor and subscript m denotes the mean value of the correlation factors P, M and F. The professional factor P accounts for the model uncertainty, calculated as the ratio of the experimental value of the ultimate capacity (P_u^{exp}) over its model prediction using specified design equations (P_u^{design}). The material factor M presents the material variability, which for steel members is mainly quantified based on the ratio of the actual yield stress (F_{ya}) to its nominal value (F_{yn}) specified by the manufacturer. Lastly, the fabrication factor F considers the variability in the section properties effective in calculating the member resistance to the action of interest, which, for the case of axial compression, is the ratio of actual cross-section area (A_{ga}) over its nominal value (A_{gn}). In addition, the coefficient c_ϕ and the coefficient of variation of resistance and load variables (V_R and V_S) are defined as follows.

$$c_\phi = \frac{\alpha_n \gamma_G + \gamma_Q}{\alpha_n \left(G_m / G_n\right) + \left(Q_m / Q_n\right)} \tag{4}$$

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \tag{5}$$

$$V_S = \frac{\sqrt{\left[\alpha_n \left(G_m / G_n\right) V_G\right]^2 + \left[\left(Q_m / Q_n\right) V_Q\right]^2}}{\alpha_n \left(G_m / G_n\right) + \left(Q_m / Q_n\right)} \tag{6}$$

where $\alpha_n = G_n/Q_n$, V_i is the coefficient of variation of variable i, and γ_G, G_n and G_m are the partial load factor and nominal and mean values for permanent gravity loading, including the self-weight and superimposed dead loads. Similarly, γ_Q, Q_n and Q_m are defined as the partial load factor and nominal and mean values for live loads, respectively.

Alternatively, Eq. (3) can be rearranged such that the strength reduction factor corresponding to a target level of reliability index (β) is obtained, viz.

$$\phi = c_{\phi} P_m M_m F_m \exp\left(-\beta\sqrt{V_R^2 + V_S^2}\right) \tag{7}$$

3.2.2 Assumptions

Based on values available from test data on CFS beams, Hsiao et al. [25] developed statistical parameters for the correlation factors P, M and F as listed in Table 2, which lead to $V_R \approx 0.14$.

Table 2: Statistical parameters for resistance factors [25]

Factor	Mean	CV
Professional (P)	1.11	0.09
Material (M)	1.10	0.10
Fabrication (F)	1.00	0.05

Following the recommendations for the minimum reliability index in the Australian standard [3] and North American specification [4] for the design of CFS members, i.e. β=2.5 corresponding to α_n=1/5, and the suggested statistical parameters for dead and live loads by Ellingwood et al. [26] as the basis for ASCE 7 [27] and AS 1170.1 [28] loading codes, the statistical parameters participating in the evaluation of the strength reduction factor can be simplified to

$$c_{\phi} = 0.826\left(0.2\gamma_G + \gamma_Q\right), \quad V_S = 0.21 \tag{8}$$

It is also noteworthy that using only numerical data would be characteristically different from a conventional reliability-based assessment using experimental data, as any numerical simulation may only approximately predict the actual behaviour depending on the variability of material, imperfections, failure modes, and the accuracy of the numerical model. Therefore, revising the professional factor P and its associated coefficient of variation V_P may seem rational when using numerical data in the validation process. Denoting $P^E = P_u^{\text{exp}}/P_u^{\text{FE}}$ and $P^M = P_u^{\text{FE}}/P_u^{\text{design}}$, one may incorporate the following adjustments for this purpose.

$$P_m = P_m^{\ E} \times P_m^{\ M}, \quad V_P \cong \sqrt{V_{PE}^2 + V_{PM}^2} \tag{9}$$

which is based on assuming that the joint variability of P^E and P^M is insignificant, i.e. cov(P^E,P^M)≈0. It is noted that P^E and its associated coefficient of variation V_{PE} depict the closeness between FE model predictions and experimental variations, which has already been established in [22]. In contrast, P^M and its associated coefficient of variation V_{PM} for a given standard design method show the correlation between the model predictions and FE results from the parametric studies. In this study, the statistical parameters for FE predictions are included in the assessment according to Eq. (9) with P_m^E=0.98 and V_{PE}=0.02.

3.2.3 Results

Using the current standard design methods, a reliability analysis is performed according to Eq. (3) to evaluate the reliability index, assuming partial and strength reduction factors consistent with each design standard, i.e. γ_m=1.0 in EC and ϕ=0.85 in AS/NZS and AISI. The results are summarised in Table 3, including the mean (μ_i) and standard deviation (σ_i) for the predictions in terms of $P_u^{\text{FE}}/P_u^{\text{design}}$, which includes values corresponding to each individual L/D data set as well as averaged values for one combined set. Despite the similarity of the DSM and EWM design equations in the AISI and AS/NZS provisions, two different reliability indices are obtained as the load factors for the ultimate gravity case in these standards differ slightly. As can be seen, the current EWM in EC3 shows a closer match to target reliability for ultimate capacity prediction of sections with predominant local buckling mode, but it provides substantially conservative predictions for the distortional buckling set. On the contrary, the EWM in AS/NZS4600 yields consistent but slightly unsafe predictions for both sets. The current DSM offers consistent but conservative predictions for both sets, which will be investigated and adjusted in the next section.

Table 3: Reliability index for built-up members using the current design equations

Mode	Variable	Direct Strength Method		Effective Width Method	
		AS4600	AISI S100	AS4600	EC3
Local	μ_L	1.10	1.10	1.02	1.23
	σ_L	0.08	0.08	0.08	0.09
	β_L	2.91	3.14	2.42	2.58
Distortional	μ_D	1.07	1.07	1.02	1.49
	σ_D	0.08	0.08	0.10	0.17
	β_D	2.77	3.00	2.43	3.25
Sectional (Combined)	μ_o	1.08	1.08	1.02	1.37
	σ_o	0.08	0.08	0.09	0.19
	β_o	2.84	3.06	2.43	2.80

Next, the required strength reduction factor (ϕ=1/γ_m) for each method is obtained according to Eq. (7) such that a reliability index of 2.5 associated with α_c=1/5 is satisfied. The results are given in Table 4, which includes separate strength reduction factors (ϕ_L, ϕ_D) for different sectional modes for comparison and one averaged value (ϕ_{LD}) for overall predictions with both L and D sets combined as having different strength reduction factors for each buckling mode is not common in design standards. Furthermore, separate reliability analyses were performed for each built-up configuration (2C, 3C, 4C), and the obtained strength reduction factors are reported in the Table to closely observe the variations in the reliability of DSM predictions for each configuration. Regarding the observed gap between the strength reduction factors obtained for each method versus its target value, the current EWM in EC3 shows the closest match for local buckling but the highest difference for distortional buckling due to its inherent conservative limitations.

As far as direct strength design equations are concerned, the strength reduction factors obtained for sectional buckling of single channel sections are in agreement with the background reliability assessment in the commentary on the AISI Specification [4], which was based on an extensive experimental data set on local and distortional buckling combined. The current DSM almost satisfies the target limit for built-up I-sections (2C) with variations in different sectional buckling modes. However, a lower strength reduction factor (ϕ≈0.8) was obtained based on the Australian practice due to the live load combination factor difference. It is noteworthy that the discrepancy between the strength reduction factors for US and AU/NZ practices is a result of using the FOSM reliability method. Unlike the American Specification, the current strength reduction factors in the Australian standards were obtained using the first-order reliability method (FORM), assuming the Gumbel distribution for live load rather than a simple log-normal distribution as in the FOSM reliability approach, and slightly higher values of reduction factors were obtained. Therefore, using the same strength reduction factor (ϕ=0.85) for US and AU/NZS practices is still valid for designing built-up sections using the current direct strength equation for distortional buckling. In addition, the strength reduction factor obtained for complex built-up sections tested in this study (3C and 4C) is found to be noticeably higher than the target limit, which is the main subject of proposed revisions in the next section.

Table 4: Strength reduction factors for built-up members using the current design equations

Mode	Section geometry	Direct Strength Method		Effective Width Method	
		AS4600	AISI S100	AS4600	EC3
Local	2C	0.85	0.90	0.79	0.99
	3C	0.90	0.96	0.84	1.03
	4C	0.93	0.98	0.87	1.04
	All	0.89	0.94	0.83	1.02
Distortional	2C	0.81	0.86	0.77	1.14
	3C	0.88	0.93	0.85	1.22
	4C	0.93	0.98	0.89	1.34
	All	0.87	0.93	0.83	1.22
Sectional (Combined)	2C	0.83	0.88	0.79	1.08
	3C	0.90	0.95	0.84	1.10
	4C	0.93	0.99	0.88	1.13
	All	0.88	0.93	0.83	1.09

4 MODIFICATION OF THE DIRECT STRENGTH DESIGN EQUATIONS

The built-up I-section and AISI specification are selected as the reference baseline for recovering the conservatism in the current DSM predictions for sectional buckling in complex sections. A meaningful dependency of ultimate capacities on s/L_{crd} was observed, which was pronounced at low distortional buckling slenderness ratios corresponding to a transition from squash sectional capacity to inelastic distortional buckling capacity. Moreover, an overall dependency on the assembly configuration of the built-up sections was observed, which is characterised in this study based on the relative number of restrained elements/flanges (n_{fr}) with respect to the total number of elements/flanges (n_{ft}) undergoing distortional buckling, i.e. $n_{rd}=n_{fr}/n_{ft}$. These two factors are chosen as the main parameters in the proposed modifications to the direct strength design equation. The definition of restrained elements is illustrated in Figure 1a for all the built-up cross-sections considered in set-D, and the corresponding values of n_{fr}, n_{ft} and n_{rd} are presented in Table 5.

Table 5: Values of the relative number of restraint elements subject to distortional buckling

Section geometry	n_{ft}	n_{fr}	n_{rd}
2C	4	0	0.00
3C	6	2	0.33
4C	8	4	0.50

In addition, the relative ratio of restrained elements in the context of local buckling (n_{rl}) refers to the relative number of major constituent elements undergoing local buckling (e.g. webs as considered here) that are partially constrained in the built-up configuration (n_{wr}) with respect to the total number of elements undergoing local buckling (n_{wt}). This ratio is equivalent to one minus the relative number of free elements (n_{fl}), i.e. $n_{fl}=1-n_{rl}$. Definition of restrained elements for local buckling is shown in Figure 1b, and the determination of n_{rl} for each cross-section is presented in Table 6. It is noted that connecting the webs of channel sections in an I-section using typical fasteners is not expected to prevent local buckling, and thus, the webs in this scenario are not counted as restrained elements for the determination of n_{rl}.

Table 6: Values of the relative number of restraint elements subject to local buckling for

Section geometry	n_{wt}	n_{wr}	n_{rl}
2C	2	0	0.00
3C	3	1	0.33
4C	4	2	0.50

According to the nature of a general direct strength curve per Eq. (1) and the dependency of its slope and pattern at different slenderness ratios on the participating parameters, the parameter a_i is kept constant, and the adjustments for built-up sections are incorporated through modification of the other two parameters (b_i, c_i). A generalised reduction function f_r in the form of a generalised nonlinear interpolation function with adjustable power is utilised for the quantification of reductions in the DSM parameters with respect to the influencing design variables:

$$f_r(\xi_i) = \chi_{r\xi} + (1-\chi_{r\xi})\xi_i^{m_\xi}, \tag{10}$$

in which ξ_i (i.e. n_{fi}=1−n_{ri} or s/L_{cri}) is the influencing design variable, $\chi_{r\xi}$ is the maximum reduction ratio for the parameter of interest when ξ_i=0 as the ideal constraint condition (i.e. $n_{ri}\rightarrow1$ or $s/L_{cri}\rightarrow0$), and m_ξ is the associated exponent that controls the rate of reduction over intermediate design values ($0<\xi_i<1$). It is noted that the term n_{ri} is generalised from the previously explained parameter n_{rd}, and it now refers to the relative number of restrained elements undergoing the sectional mode of interest *i*. For the case of m_ξ=1, the function (f_r) is the simple linear interpolation between $\chi_{r\xi}$ and 1. This form for the proposed reduction function is selected so that the proposed expressions for different sectional buckling modes (*i*=L or D) can be unified. It also somewhat resembles the participating modification function in the general form of the direct strength equations $a_i-b_i/\lambda_i^{c_i}$, where the slenderness (λ_i) can be considered as $\lambda_i=1/\xi_i$, and the other parameters are defined as $a_i=\chi_{r\xi}$, $b_i=a_i-1=\chi_{r\xi}-1$, $c_i=m_\xi$. This highlights another rational feature of the proposed reduction function over an arbitrary regression, which is its gradual reduction with an adjustable rate from an initial value of unity (i.e. non-composite case) towards an asymptote ($\chi_{r\xi}$) as the reduction factor at high slenderness ratios ($\lambda_i\rightarrow\infty$).

By adopting the generalised reduction function, the following unified adjustment expression is proposed for the revision of the current DSM parameters in either of the sectional buckling modes:

$$b_{ir} = b_{i0} f_r(s/L_{cri})^{1/f_r(1-n_{ri})}, \tag{11}$$

$$c_{ir} = c_{i0} f_r(1-n_{ri})^{1/f_r(s/L_{cri})}, \tag{12}$$

where b_{i0} and c_{i0} refer to the current values of these parameters in the relevant DSM design equation prior to any adjustments, as presented in Table 1. This function contains a primary reduction function for each DSM parameter depending on the primary influencing design variable (e.g. 1−n_{ri} for c_{ir}), which is further modified using the secondary design variable (e.g. s/L_{cri} for c_{ir}) by adjusting the exponent that regulates the rate of enhancement. The regression parameters for the adjusted DSM variables are back-calculated through nonlinear regression analysis to test data at different slenderness ratios to meet the target strength reduction factor for DSM predictions, and are summarised in Table 7.

Table 7: Regression parameters for the adjustment of DSM variables

Design variable (ξ)	maximum reduction ($\chi_{r\xi}$)	exponent (m_ξ)
n_{fi}=1−n_{ri}	0.75	0.8
s/L_{crd}	0.70	0.8
s/L_{crl}	0.70	4.0

Following the proposed modifications, the DSM equations for sectional buckling of built-up columns are revised as follows.

$$\frac{P_{ni}}{P_y} = \begin{cases} 1 & \forall \lambda_i \le \lambda_{i.\lim}, \\ \left(1 - \frac{b_{ir}}{\lambda_i^{c_{ir}}}\right) \frac{1}{\lambda_i^{c_{ir}}} & \forall \lambda_i > \lambda_{i.\lim}, \end{cases} \tag{13}$$

where i is the sectional mode of interest, i.e. distortional (d) or local (l), and

$$\lambda_{i.\lim} = \left[\frac{1 - \sqrt{1 - 4b_{ir}}}{2b_{ir}}\right]^{\frac{-1}{c_{ir}}}, \tag{14}$$

$$b_{dr} = 0.25\left(0.7 + 0.3\left(s/L_{crd}\right)^{0.8}\right)^{\frac{1}{0.75 + 0.25\left(1 - n_{fd}\right)^{0.8}}}, \tag{15}$$

$$c_{dr} = 1.2\left(0.75 + 0.25\left(1 - n_{fd}\right)^{0.8}\right)^{\frac{1}{0.7 + 0.3\left(s/L_{crd}\right)^{0.8}}}, \tag{16}$$

$$b_{lr} = 0.15\left(0.7 + 0.3\left(s/L_{crl}\right)^{4}\right)^{\frac{1}{0.75 + 0.25\left(1 - n_{fl}\right)^{0.8}}}, \tag{17}$$

$$c_{lr} = 0.8\left(0.75 + 0.25\left(1 - n_{fl}\right)^{0.8}\right)^{\frac{1}{0.7 + 0.3\left(s/L_{crl}\right)^{4}}}, \tag{18}$$

in which s/L_{cri} shall not be taken as more than 1.0.

Upon the systematic adjustment of direct strength design equations with parametric variables, the strength reduction factor and reliability index for more complex built-up sections were successfully restored to levels consistent with baseline built-up I-sections as commonly used and verified in building research and practice. A comparison of the accuracy and scatter of the DSM predictions prior to and after proposed adjustments are made in Figure 4 and Figure 5 for distortional and local buckling, respectively. The results of the proposed DSM show a substantial improvement in the accuracy of ultimate load predictions for different built-up section configurations.

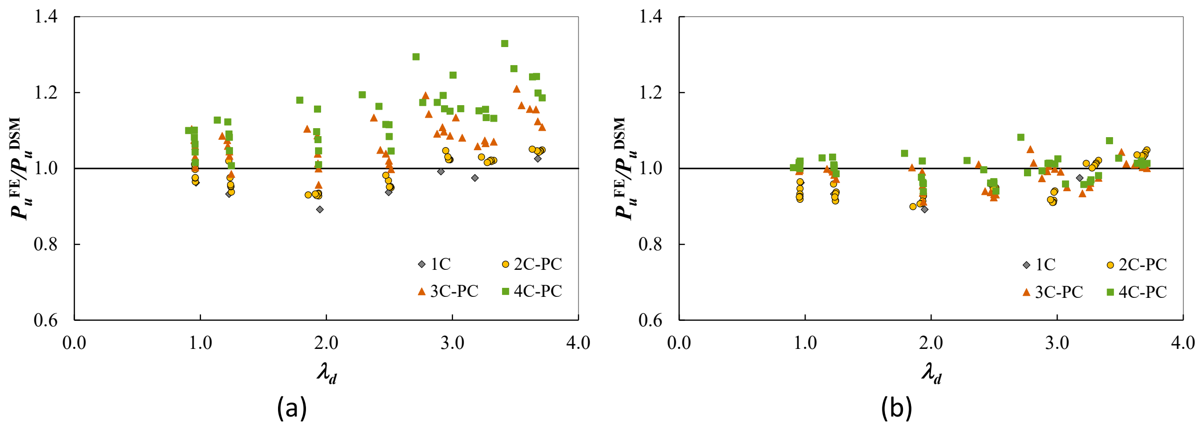
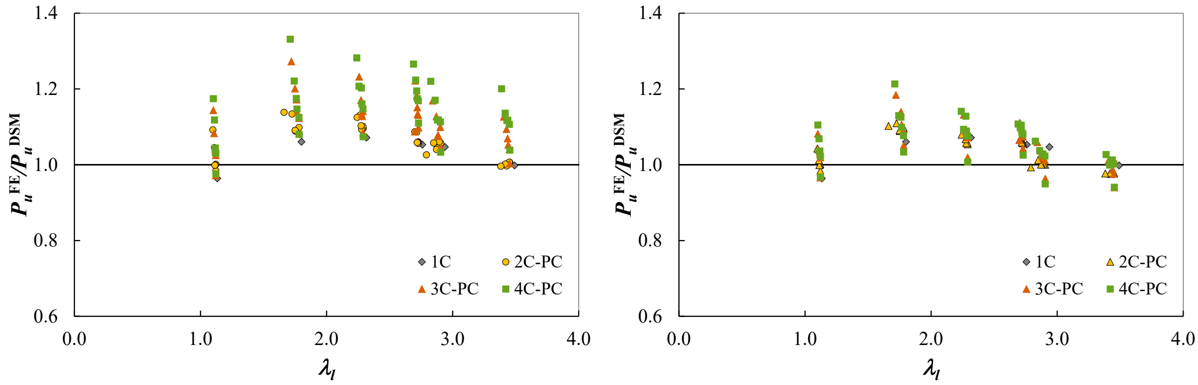


Figure 4: Scatter of data points for ultimate distortional buckling capacity around model predictions based on the (a) current DSM, (b) proposed DSM



(a) (b)

Figure 5: Scatter of data points for ultimate local buckling capacity around model predictions based on the (a) current DSM, (b) proposed DSM

5 CONCLUSIONS

The results from extensive parametric studies carried out in [22] were utilised to perform a reliability-based assessment, where the suitability and performance of the two design standard methods, namely the traditional Effective Width Method and the Direct Strength Method, in the European, North American and Australian/New Zealand standards were evaluated for the design of built-up CFS compression members under sectional buckling. The reliabilities of each design method were highlighted based on the outcomes of the assessment. The effective width method was found to provide the closest reliability to the target values for the local buckling capacity; however, its application is not straightforward and is further limited by several slenderness limitations in place. On the contrary, the direct strength method as a simple alternative approach shows superior performance for distortional buckling while still showing safe and reasonable predictions for local buckling. The DSM, in combination with the compound strip method for elastic buckling analysis, was found to be applicable to built-up members and a wider range of sections compared to the EWM, which promotes its use even as the default design method due to its simplicity and extended range of applicability.

A rational modification of the DSM design parameters for local and distortional buckling capacities of built-up columns was proposed based on the relative number of restrained components (n_{ri}) undergoing the sectional buckling mode of interest and the normalised fastener spacing ratio in terms of the associated critical buckling half-wavelength (s/L_{cri}). The proposed modifications were shown to be reasonably accurate and robust for different cross-section assemblies and fastener configurations and to fairly accurately account for the strength enhancement produced by intermittent fasteners.

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