International Specialty Conference on Cold-Formed Steel Structures

(2006) - 18th International Specialty Conference on Cold-Formed Steel Structures

Oct 26th, 12:00 AM

Direct Strength Method for Lipped Channel Columns and Beams Affected

Nuno Silvestre

Eighteenth International Specialty Conference on Cold-Formed Steel Structures
Orlando, Florida, U.S.A, October 26 & 27, 2006

Direct Strength Method for Lipped Channel Columns and Beams Affected by Local-Plate/Distortional Interaction

Nuno Silvestre[1], Pedro B. Dinis[1] and Dinar Camotim[2]

Abstract

This paper reports the results of an investigation on the use of the Direct Strength Method (DSM) to estimate the ultimate strength of lipped channel cold-formed steel columns and beams affected by interaction phenomena involving local-plate and distortional buckling modes. Initially, one briefly presents the DSM approaches to safety check columns and beams against local-plate and distortional failures, and some attention is also devoted to a recently proposed extension aimed at accounting for the above buckling mode interaction. Next, one describes the results of a parametric study, carried out in code ABAQUS, to determine the "exact" ultimate strengths of *108 columns* and *90 beams* displaying various cross-section dimensions and lengths, all selected to ensure the occurrence of relevant mode interaction effects. Then, these ultimate strength data are compared with the estimates provided by the existing DSM equations and, on the basis of this comparison, one identifies some features that must necessarily be included in a DSM approach that properly accounts for local-plate/distortional interaction.

Introduction

The Direct Strength Method (DSM) was originally proposed by Schafer & Peköz (1999) about six years ago and has been continuously improved ever

[1]Assistant Professor, [2]Associate Professor, Civil Eng. Dept., IST/ICIST, Technical University of Lisbon, Av. Rovisco Pais, 1049-001 Lisboa, Portugal

since, mainly due to the efforts of Schafer (2002, 2003a,b). Moreover, note the very recent inclusion of the DSM in the NAS and AS/NZS cold-formed steel design specifications – it already appears in the current (new) versions of these codes (AISI 2004, SA-SNZ 2005). The method provides an elegant, efficient and consistent approach to estimate the ultimate strength of cold-formed steel columns and beams (i) exhibiting global (flexural, torsional or flexural-torsional), local-plate or distortional collapses, or (ii) failing in mechanisms that involve interaction between local-plate and global buckling modes. Indeed, the most recent DSM version prescribes the need to perform two independent safety checks, regardless of the member critical buckling mode nature: (i) one against a pure distortional collapse and (ii) another against failure in a pure local-plate mode (laterally braced members) or due to local-plate/global mode interaction. In the latter case, the DSM is a very efficient alternative to the classical "effective width method".

However, as pointed out by Schafer (2002, 2003b, 2005), further research is needed before the DSM approach can be successfully applied to members (i) under compression and bending (Duong & Hancock 2004, Rasmussen & Hossain 2004) or (ii) affected by interaction phenomena involving distortional buckling modes (Yang & Hancock 2004, Kwon *et al.* 2005). Since it has been recently shown that coupling between local-plate and distortional buckling modes may strongly influence the post-buckling behavior and ultimate strength of commonly used lipped channel cross-sections (Dinis *et al.* 2005a,b), it would be obviously convenient to have this mode interaction phenomenon also covered by the DSM.

The aim of this work is to contribute towards extending the current DSM domain of application, by making it possible to predict ultimate strengths of lipped channel columns and beams affected by local-plate/distortional buckling mode interaction. To achieve this goal, one begins by carrying out an extensive parametric study involving the evaluation of the elastic-plastic failure loads/moments of lipped channel columns/beams with distinct cross-section dimensions, lengths and yield stresses, and containing critical-mode small-amplitude initial geometrical imperfections – the member geometries were carefully selected to ensure the occurrence of local-plate/distortional interaction. All second-order elastic-plastic analyses were carried out in the finite element code ABAQUS (HKS 2002), adopting 4-node shell elements to discretize the members. These ultimate strength values then provide a "data bank" enabling the proposal and validation of preliminary recommendations on the use of a DSM approach to estimate the ultimate strength of lipped channel columns and beams against local-plate/distortional mode interaction.

The Direct Strength Method (DSM)

Compared with the classical "*effective width* approach", the DSM exhibits three major innovative features, all due to the fact that the cross-section is treated as a *whole*: (i) wall-restraint effects are always taken into account, (ii) no effective width calculations are needed and (iii) strength estimates are provided for member *distortional* collapses. Moreover, the DSM provides a rational and systematic approach to design thin-walled members with arbitrary cross-sections, loadings or failure modes – of course, a given application must be properly calibrated and validated (comparison with experimental and/or numerical results). Finally, note that the DSM and effective width approaches share one basic assumption: the member ultimate strength can be accurately predicted solely on the basis of its elastic buckling and yield stresses.

The current DSM approach adopts "Winter-type" design curves, which were calibrated against a large number of experimental and/or numerical results (Schafer 2003a). It was shown that, when a given member fails in pure local-plate or distortional modes, safe and accurate ultimate strength estimates can be obtained on the basis of elastic buckling and yield stress values only. Thus, the DSM stipulates that the *nominal strengths*, against *local-plate* and *distortional* failure, of laterally braced columns (P_{nl} and P_{nd}) and beams (M_{nl} and M_{nd}) are yielded by the expressions (Schafer 2002, Hancock *et al.* 2001)

$$\begin{aligned} P_{nl} &= P_y \quad if \quad \lambda_l \leq 0.776 \\ P_{nl} &= P_y \left(\frac{P_{crl}}{P_y}\right)^{0.4} \left[1-0.15\left(\frac{P_{crl}}{P_y}\right)^{0.4}\right] \quad if \quad \lambda_l > 0.776 \end{aligned} \quad , \quad (1)$$

$$\begin{aligned} P_{nd} &= P_y \quad if \quad \lambda_d \leq 0.561 \\ P_{nd} &= P_y \left(\frac{P_{crd}}{P_y}\right)^{0.6} \left[1-0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right] \quad if \quad \lambda_d > 0.561 \end{aligned} \quad , \quad (2)$$

$$M_{nl} = M_y \quad if \quad \lambda_l \leq 0.776$$
$$M_{nl} = M_y \left(\frac{M_{crl}}{M_y}\right)^{0.4} \left[1 - 0.15\left(\frac{M_{crl}}{M_y}\right)^{0.4}\right] \quad if \quad \lambda_l > 0.776 \quad , \quad (3)$$

$$M_{nd} = M_y \quad if \quad \lambda_d \leq 0.673$$
$$M_{nd} = M_y \left(\frac{M_{crd}}{M_y}\right)^{0.5} \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right] \quad if \quad \lambda_d > 0.673 \quad , \quad (4)$$

where (i) one has $\lambda_l=(P_y/P_{crl})^{0.5}$ or $\lambda_l=(M_y/M_{crl})^{0.5}$ and $\lambda_d=(P_y/P_{crd})^{0.5}$ or $\lambda_d=(M_y/M_{crd})^{0.5}$, (ii) P_y and M_y are the squash load and plastic moment, (iii) P_{crl} (M_{crl}) and P_{crd} (M_{crd}) are the *local-plate* and *distortional* critical buckling loads (moments). In order to capture the local-plate/global interaction (in laterally unbraced members), the DSM approach replaces P_y by P_{ne} in eqs. (1) and M_y by M_{ne} in eqs. (3), where P_{ne} and M_{ne} are the column/beam buckling strengths associated with *global* failure (Hancock *et al.* 2001). Note that it is important to predict accurately the column (beam) distortional failure load (moment), since (i) the distortional post-critical strength is considerably lower and more imperfection-sensitive than its local-plate counterpart (*e.g.*, Dinis & Camotim 2004) and (ii) there is clear (numerical) evidence that columns and beams buckling in local-plate modes often exhibit distortional failure mechanisms (Schafer & Peköz 1999, Dinis & Camotim 2004).

Parametric Study: Scope, Numerical Analysis and Results

Scope. In order to be able to carry out a rather large parametric study on the ultimate strength of lipped channel columns and beams affected by local-plate/distortional interaction, their geometries had to be carefully selected: it was necessary to find sets of cross-section dimensions and lengths making it possible to "control" the closeness between the column/beam local-plate and distortional critical buckling stresses (σ_{crl} and σ_{crd} – in beams, σ is the uniform flange applied stress). This goal was achieved through a trial-and-error approach to find *12* "basic cross-section shapes" (*6* for the columns and *6* for the beams) with commonly used dimensions and ensuring that σ_{crl} and σ_{crd} coincide. The search led to the following columns and beams:

(i) Three *slender* columns ($1.4 \leq \lambda_d \leq 2.6$),
(i_1) $b_w=100$, $b_f=50$, $b_s=5$ and $t=1.0mm$, $L=270mm$.
(i_2) $b_w=120$, $b_f=80$, $b_s=10$ and $t=1.3mm$, $L=550mm$.
(i_3) $b_w=95$, $b_f=80$, $b_s=10$ and $t=0.95mm$, $L=600mm$.
Three *stockier* columns ($0.6 \leq \lambda_d \leq 1.4$),
(i_4) $b_w=180$, $b_f=100$, $b_s=20$ and $t=3.4mm$, $L=650mm$.
(i_5) $b_w=110$, $b_f=78$, $b_s=30$ and $t=2.8mm$, $L=800mm$.
(i_6) $b_w=100$, $b_f=100$, $b_s=26$ and $t=2.0mm$, $L=950mm$.

(ii) Three *slender* beams ($1.4 \leq \lambda_d \leq 2.6$),
(ii_1) $b_w=180$, $b_f=70$, $b_s=15$ and $t=1.1mm$, $L=750mm$.
(ii_2) $b_w=400$, $b_f=150$, $b_s=26$ and $t=2.0mm$, $L=1400mm$.
(ii_3) $b_w=390$, $b_f=100$, $b_s=12$ and $t=1.4mm$, $L=750mm$.
Three *stockier* beams ($0.6 \leq \lambda_d \leq 1.4$),
(ii_4) $b_w=120$, $b_f=75$, $b_s=24$ and $t=1.8mm$, $L=770mm$.
(ii_5) $b_w=160$, $b_f=80$, $b_s=23$ and $t=1.7mm$, $L=820mm$.
(ii_6) $b_w=80$, $b_f=50$, $b_s=10$ and $t=0.8mm$, $L=450mm$.
Note that all these columns/beams satisfy the cross-section dimension requirements ("pre-qualified columns") adopted in the DSM approach.

(iii) Subsequently, the closeness between σ_{crl} and σ_{crd} was slightly altered, by just changing a single basic cross-section dimension: flange width b_f, web width b_w or stiffener width b_s. This procedure made it possible to identify various members with (iii_1) cross-section dimensions generated from the basic shapes and (iii_2) very close (but not necessarily equal) σ_{crl} and σ_{crd} values – they are all such that $0.85 \leq \sigma_{crl}/\sigma_{crd} \leq 1.20$.

The member lengths considered always correspond to single distortional half-waves associated with the buckling stresses σ_{crd} and were obtained through finite strip analyses. The steel behavior is characterized by $E=210\,GPa$ (Young's modulus), $\nu=0.3$ (Poisson's ratio) and $f_y=250\text{-}350\text{-}550\ MPa$ (columns), and $f_y=250\text{-}350\text{-}450\,MPa$ (beams) – these yield stresses meet the DSM limit for "pre-qualified columns and beams". Finally, it is worth (i) noting that no residual stresses have been accounted for and (ii) addressing the criterion adopted to select the initial geometrical imperfections included in the non-linear analyses that provide the column and beam ultimate strengths:

(i) Regardless of their critical stress ratios $\sigma_{crl}/\sigma_{crd}$, all the columns and beams analyzed contained initial geometrical imperfections with a single-wave distortional buckling mode shape, having an amplitude (mid-span compressed flange-stiffener corner displacement) equal to *10%* of the wall thickness *t* and involving either *outward* (columns) or *inward* (beams) flange-stiffener assembly motions – recent studies, involving lipped channel columns and beams with $\sigma_{crl}=\sigma_{crd}$, showed that these imperfection shapes are the *most detrimental* ones, since they correspond to the lowest post-buckling strength and collapse loads and moments (Dinis *et al.* 2005, Martins 2006).

(ii) The *slender* columns (i_1-i_3) (not the beams) with $\sigma_{crl}/\sigma_{crd} < 1.0$ (*i.e.*, that buckle in *local-plate* modes with several half-waves) were also analyzed in the presence of *critical-mode* initial imperfections, again with amplitude $0.1\,t$ – now the mid-web flexural displacement at mid-span.

A total of (i) *66 slender* and *45 stocky* columns, and (ii) *45 slender* and *45 stockier* beams were analyzed, corresponding to all possible combinations of *16* (column) and *15* (beam) different cross-section shapes and *3* yield stress values. All the cross-section dimensions (b_w, b_f, b_s, t), length values (L), yield stresses (f_y) and initial imperfection shapes (D, LP), considered in this work, as well as the corresponding critical stresses (σ_{crl}, σ_{crd}), are given in tables 1A-C (columns) and 2A-C (beams) and will be addressed further ahead.

Numerical analysis. This subsection deals with the *numerical* evaluation of the "*exact*" column and beam ultimate strengths, which are subsequently used to assess the merits of the DSM approach described before. These ultimate strengths were obtained by means of *finite element* analyses (FEA) carried out in the code ABAQUS (HKS 2002), discretizing the members into *shell* elements. As far as the performance of these FEA is concerned, the following aspects deserve to be mentioned here (Dinis & Camotim 2006):

(I) *Discretization*. The member mid-surfaces were discretized into S4 finite elements (ABAQUS nomenclature: isoparametric 4-node shell elements with the shear stiffness yielded by a *full* integration rule), which were found to be the most adequate to carry out this task. One considered *20-30* elements along the cross-section mid-line (width of about *10 mm*) and previous convergence/accuracy studies showed that the element length-to-width ratio should be comprised between *1* and *2*.

(II) *Support Conditions*. All member end sections are locally and globally pinned and can warp freely. Concerning the first aspect, these support conditions were modeled by imposing null transverse membrane and flexural displacements at all end section nodes – in order to preclude the occurrence of a spurious longitudinal rigid-body motion, the axial displacement was prevented at one mid-span cross-section node.

(III) *Column Loading*. Compressive forces, equivalent to a uniform normal stress distribution, are applied at the nodes of the column end-sections. Since the reference value of the *load parameter p* is $t\,N/mm$ (t – wall thickness), which corresponds to a $1\,MPa$ uniform stress distribution, the value of p yielded by ABAQUS is numerically equal to the *average stress* acting on the column (expressed in *MPa*).

(IV) *Beam Loading*. Compressive and tensile forces $p=\sigma t$, equivalent to the linearly varying normal stress distribution due to a bending moment, were applied at the nodes of the beam end-sections. Since the reference value of the *load parameter p* corresponds to a $1\,MPa$ flange stress, the value of p yielded by ABAQUS is numerically equal to the *average stress* acting on the beam flanges (expressed in *MPa*).

(V) *Material Modeling*. The member (carbon steel) material behavior, deemed isotropic and homogeneous, was modeled through (i) linear elastic (bifurcation analysis) and (ii) elastic/perfectly-plastic (post-buckling analysis) stress-strain laws. In the latter case, the well-known Prandtl-Reuss model (J_2-flow theory), combining Von Mises's yield criterion with the associated flow rule, was adopted. These stress-strain laws are readily available in the ABAQUS material behavior library and one just needs to provide the values of E, ν and f_y.

(VI) *Initial Imperfections*. All initial geometrical imperfections, defined earlier (buckling mode shapes with amplitude $0.1\,t$) are included in the analyses through a specific ABAQUS command. In columns/beams that buckle in local-plate modes ($\sigma_{crl}<\sigma_{crd}$), the single-wave "distortional" imperfection is, effectively, the column higher-order buckling mode most resembling it, which means that it is not possible to guarantee the "purity" of the distortional shape – *e.g.*, the small participation of a multiple half-wave local-plate mode is virtually undetectable.

Numerical results. The numerical results included in tables 1A-C (columns) and 2A-C (beams) consist of (i) local-plate and distortional bifurcation stresses and (ii) average stresses at collapse (σ_u). In order to better convey the meaning of these results, they are illustrated in figure 1(a), which shows the post-buckling equilibrium paths (σ/σ_{cr} *vs.* v/t) of columns with (i) $\sigma_{crl}=\sigma_{crd}$ ($\equiv\sigma_{cr}$), (ii) identical outward distortional imperfections and (iii) four distinct yield stress values: $f_y/\sigma_{cr}\approx 1.2, 2.0, 3.5, 5.5$. It is worth noting that the onset of yielding, always taking place in the stiffener free ends (see figure 1(b_1)), occurs at the equilibrium points A and may or may not trigger the column failure – it depends on the f_y/σ_{cr} ratio. Indeed, for large enough f_y/σ_{cr} values failure occurs at a limit point B, following (i) a "snap-through" phenomenon and (ii) the yielding of the column central regions located around the web-flange corners – see figure 1(b_2) (Dinis *et al.* 2005a,b). As for figure 1(c), it shows the corresponding (predominantly distortional) failure mechanism.

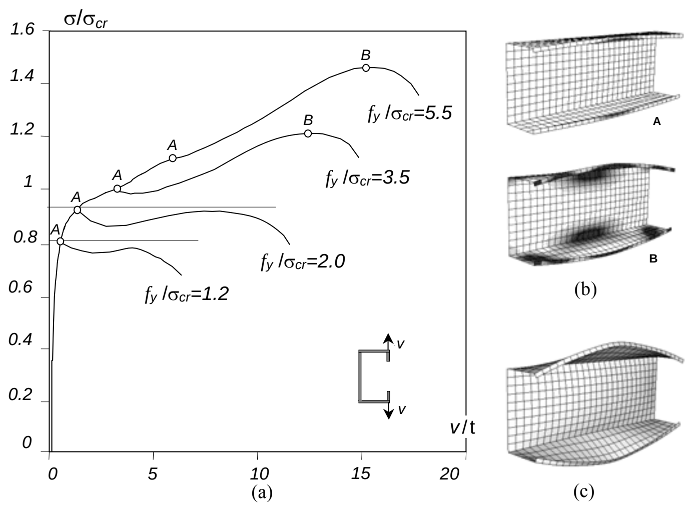

Fig. 1: (a) Post-buckling equilibrium paths, (b) plastic strain distributions and (c) failure mechanism

Assessment of the DSM Estimates

The numerical and DSM results concerning the *108 columns* and *90 beams* analyzed, which are presented in tables 1A-C and 2A-C, make it possible to compare the "exact" ultimate strengths (σ_u) with their DSM estimates (σ_{nd} and σ_{nl}). The observation of these results prompts the following remarks:

(i) The *column* σ_u values concerning the local-plate imperfections (their varying dimensions exhibit the superscript ^{LP} – tables 1A-B) are never below their distortional counterparts, thus confirming the assertion made earlier: the distortional imperfections are the *most detrimental* ones

Table 1A. Comparison between the "exact" ultimate strengths and their DSM estimates (σ_{nl}, σ_{nd} and σ_{nld}) for *42* (out of *108*) columns

			FEA			DSM					
	b_f	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=100, b_s=5, t=1.0, L=270mm	55	250	101	91	94	156	1.66	117	1.24	95	1.01
	55	350	101	91	110	193	1.75	138	1.25	106	0.96
	55	550	101	91	131	258	1.97	171	1.31	121	0.92
	52.5	250	101	96	97	156	1.61	121	1.25	97	1.00
	52.5	350	101	96	114	194	1.70	143	1.25	108	0.95
	52.5	550	101	96	137	258	1.88	176	1.28	124	0.91
	50	250	102	102	102	156	1.53	125	1.23	99	0.97
	50	350	102	102	120	194	1.62	147	1.23	110	0.92
	50	550	102	102	147	259	1.76	182	1.24	127	0.86
	47.5	250	102	108	107	157	1.47	128	1.20	101	0.94
	47.5	350	102	108	127	194	1.53	151	1.19	113	0.89
	47.5	550	102	108	156	259	1.66	187	1.20	130	0.83
	45	250	103	113	115	157	1.37	131	1.14	103	0.90
	45	350	103	113	136	195	1.43	155	1.14	115	0.85
	45	550	103	113	168	260	1.55	193	1.15	132	0.79
	47.5^{LP}	250	102	108	118	157	1.33	128	1.08	101	0.86
	47.5^{LP}	350	102	108	127	194	1.53	151	1.19	113	0.89
	47.5^{LP}	550	102	108	157	259	1.65	187	1.19	130	0.83
	45^{LP}	250	103	113	128	157	1.23	131	1.02	103	0.80
	45^{LP}	350	103	113	142	195	1.37	155	1.09	115	0.81
	45^{LP}	550	103	113	168	260	1.55	193	1.15	132	0.79
	b_w	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_f=80, b_s=10, t=1.3, L=550mm	130	250	100	110	105	155	1.48	129	1.23	101	0.96
	130	350	100	110	107	193	1.80	153	1.43	113	1.06
	130	550	100	110	123	257	2.09	189	1.54	130	1.06
	125	250	107	113	107	159	1.49	131	1.22	104	0.97
	125	350	107	113	109	198	1.82	155	1.42	116	1.06
	125	550	107	113	123	264	2.15	192	1.56	134	1.09
	120	250	115	115	109	163	1.50	133	1.22	108	0.99
	120	350	115	115	111	203	1.83	157	1.41	120	1.08
	120	550	115	115	124	271	2.19	194	1.56	139	1.12
	115	250	125	118	112	168	1.50	134	1.20	111	0.99
	115	350	125	118	114	209	1.83	159	1.39	124	1.09
	115	550	125	118	122	278	2.28	197	1.61	143	1.17
	110	250	135	121	114	172	1.51	136	1.19	115	1.01
	110	350	135	121	116	214	1.84	161	1.39	129	1.11
	110	550	135	121	121	287	2.37	199	1.64	149	1.23
	100	250	157	127	119	182	1.53	139	1.17	123	1.03
	100	350	157	127	122	226	1.85	164	1.34	138	1.13
	100	550	157	127	126	303	2.40	204	1.62	159	1.26
	125^{LP}	250	107	113	119	159	1.34	131	1.10	104	0.87
	125^{LP}	350	107	113	120	198	1.65	155	1.29	116	0.97
	125^{LP}	550	107	113	122	264	2.16	192	1.57	134	1.10

Table 1B. Comparison between the "exact" ultimate strengths and their DSM estimates (σ_{nl}, σ_{nd} and σ_{nld}) for *36* (out of *108*) columns

			FEA			DSM					
	b_s	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=95, b_f=80, t=0.95, L=600mm	11	250	92	100	94	150	1.60	123	1.31	95	1.01
	11	350	92	100	95	187	1.97	145	1.53	106	1.12
	11	550	92	100	99	249	2.52	180	1.82	122	1.23
	10.5	250	91	96	91	150	1.65	121	1.33	94	1.03
	10.5	350	91	96	92	187	2.03	142	1.54	104	1.13
	10.5	550	91	96	97	249	2.57	176	1.81	120	1.24
	10	250	91	91	86	150	1.74	118	1.37	92	1.07
	10	350	91	91	87	187	2.15	139	1.60	103	1.18
	10	550	91	91	92	249	2.71	171	1.86	118	1.28
	9.5	250	91	87	83	150	1.81	115	1.39	91	1.10
	9.5	350	91	87	84	187	2.23	135	1.61	101	1.20
	9.5	550	91	87	91	248	2.73	167	1.84	116	1.27
	9	250	91	83	78	150	1.92	112	1.44	89	1.14
	9	350	91	83	79	186	2.35	132	1.67	99	1.25
	9	550	91	83	84	248	2.95	162	1.93	114	1.36
	10.5^{LP}	250	91	96	109	150	1.38	121	1.11	94	0.86
	10.5^{LP}	350	91	96	109	187	1.72	142	1.30	104	0.95
	10.5^{LP}	550	91	96	109	249	2.28	176	1.61	120	1.10
	11^{LP}	250	92	100	114	150	1.32	123	1.08	95	0.83
	11^{LP}	350	92	100	114	187	1.64	145	1.27	106	0.93
	11^{LP}	550	92	100	114	249	2.18	180	1.58	122	1.07
	b_f	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=180, b_s=20, t=3.4, L=650mm	90	250	361	399	240	239	1.00	221	0.92	220	0.92
	90	350	361	399	298	301	1.01	276	0.93	256	0.86
	90	550	361	399	361	406	1.12	360	1.00	306	0.85
	95	250	358	377	231	239	1.03	218	0.94	217	0.94
	95	350	358	377	287	300	1.05	270	0.94	252	0.88
	95	550	358	377	341	405	1.19	351	1.03	300	0.88
	100	250	355	355	222	238	1.07	213	0.96	213	0.96
	100	350	355	355	276	299	1.08	264	0.96	247	0.89
	100	550	355	355	323	404	1.25	342	1.06	294	0.91
	105	250	353	338	217	238	1.10	210	0.97	210	0.97
	105	350	353	338	267	298	1.12	259	0.97	243	0.91
	105	550	353	338	307	403	1.31	334	1.09	289	0.94
	110	250	350	321	211	237	1.12	206	0.98	206	0.98
	110	350	350	321	256	298	1.16	253	0.99	239	0.93
	110	550	350	321	292	402	1.38	326	1.12	284	0.97

(Silvestre *et al.* 2005a,b). Since the DSM cannot capture the initial imperfection effect, its estimates should preferably approximate well the σ_u values related to distortional imperfections. If this is the case, then the DSM underestimates σ_u for columns with local-plate imperfections.

Table 1C. Comparison between the "exact" ultimate strengths and their DSM estimates (σ_{nl}, σ_{nd} and σ_{nld}) for *36* (out of *108*) columns and overall results

			FEA			DSM					
	b_w	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_f=78, b_s=30, t=2.8, L=800mm	100	250	736	656	249	250	1.00	247	0.99	247	0.99
	100	350	736	656	345	350	1.01	324	0.94	324	0.94
	100	550	736	656	508	514	1.01	442	0.87	442	0.87
	105	250	680	641	249	250	1.00	246	0.99	246	0.99
	105	350	680	641	344	350	1.02	322	0.94	322	0.94
	105	550	680	641	503	501	1.00	438	0.87	429	0.85
	110	250	630	625	248	250	1.01	246	0.99	246	0.99
	110	350	630	625	344	350	1.02	320	0.93	320	0.93
	110	550	630	625	499	489	0.98	434	0.87	416	0.83
	115	250	581	611	248	250	1.01	245	0.99	245	0.99
	115	350	581	611	342	350	1.02	318	0.93	318	0.93
	115	550	581	611	493	476	0.97	430	0.87	403	0.82
	120	250	538	596	248	250	1.01	244	0.98	244	0.98
	120	350	538	596	341	342	1.00	316	0.93	316	0.93
	120	550	538	596	489	464	0.95	426	0.87	391	0.80
	b_s	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=100, b_f=100, t=2.0, L=950mm	22	250	317	285	226	230	1.02	197	0.87	195	0.86
	22	350	317	285	262	288	1.10	241	0.92	224	0.85
	22	550	317	285	276	388	1.41	308	1.12	265	0.96
	24	250	317	299	227	230	1.01	201	0.89	198	0.87
	24	350	317	299	270	288	1.07	246	0.91	227	0.84
	24	550	317	299	287	388	1.35	315	1.10	268	0.93
	26	250	317	314	230	229	1.00	205	0.89	200	0.87
	26	350	317	314	279	288	1.03	251	0.90	230	0.82
	26	550	317	314	300	388	1.29	323	1.08	273	0.91
	28	250	316	331	232	229	0.99	208	0.90	202	0.87
	28	350	316	331	288	288	1.00	257	0.89	233	0.81
	28	550	316	331	316	388	1.23	331	1.05	277	0.88
	30	250	315	350	234	229	0.98	212	0.91	205	0.88
	30	350	315	350	297	287	0.97	263	0.89	237	0.80
	30	550	315	350	337	387	1.15	339	1.01	281	0.83
						Av.	**1.52**	**Av.**	**1.20**	**Av.**	**0.97**
						Sd.	**0.482**	**Sd.**	**0.268**	**Sd.**	**0.129**

(ii) The ratios between the predicted and "exact" *column* ultimate strength values σ_{nl}/σ_u and σ_{nd}/σ_u are often much higher than *1.0*. In fact, the DSM provisions for local-plate and distortional failure yield estimates *52%* and *20%* higher than the "exact" values (in average). Moreover, the σ_{nl}/σ_u and σ_{nd}/σ_u values are also very scattered (standard deviations of *0.48* and *0.27*) – *i.e.*, σ_{nl} and σ_{nd} considerably overestimate σ_u in *columns* affected by local-plate/distortional mode interaction.

Table 2A. Comparison between the "exact" ultimate strengths and their DSM estimates (σ_{nl}, σ_{nd} and σ_{nld}) for *30* (out of *90*) beams

			FEA			DSM					
	b_f	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=120, b_s=24, t=1.8, L=770mm	55	250	898	889	253	250	0.99	250	0.99	250	0.99
	55	350	898	889	344	350	1.02	350	1.02	350	1.02
	55	450	898	889	419	450	1.07	437	1.04	437	1.04
	65	250	692	677	249	250	1.00	250	1.00	250	1.00
	65	350	692	677	327	350	1.07	338	1.03	338	1.03
	65	450	692	677	393	439	1.12	403	1.03	403	1.01
	75	250	535	542	239	250	1.05	249	1.04	249	1.04
	75	350	535	542	311	341	1.10	316	1.02	316	1.00
	75	450	535	542	370	405	1.09	375	1.01	357	0.94
	85	250	423	454	230	250	1.09	237	1.03	237	1.03
	85	350	423	454	296	316	1.07	299	1.01	284	0.94
	85	450	423	454	347	375	1.08	352	1.02	318	0.90
	95	250	341	389	223	235	1.05	226	1.01	219	0.97
	95	350	341	389	282	295	1.05	283	1.00	256	0.90
	95	450	341	389	304	349	1.15	333	1.09	285	0.93
	b_s	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=160, b_f=80, t=1.7, L=820mm	19	250	410	345	226	249	1.10	218	0.96	218	0.96
	19	350	410	345	281	313	1.11	272	0.97	264	0.90
	19	450	410	345	315	371	1.18	318	1.01	294	0.89
	21	250	410	368	227	249	1.10	222	0.98	222	0.98
	21	350	410	368	285	313	1.10	278	0.98	268	0.91
	21	450	410	368	323	371	1.15	326	1.01	299	0.89
	23	250	409	394	228	249	1.09	227	1.00	227	0.99
	23	350	409	394	289	313	1.08	285	0.99	272	0.92
	23	450	409	394	331	371	1.12	334	1.01	304	0.89
	25	250	408	423	229	249	1.09	232	1.01	232	1.01
	25	350	408	423	292	313	1.07	292	1.00	276	0.93
	25	450	408	423	338	370	1.10	343	1.02	309	0.90
	27	250	406	455	230	248	1.08	237	1.03	237	1.03
	27	350	406	455	296	312	1.05	299	1.01	281	0.94
	27	450	406	455	345	370	1.07	352	1.02	314	0.90

(iii) The ratios between the predicted and "exact" *beam* ultimate strength values σ_{nl}/σ_u and σ_{nd}/σ_u are also often quite higher than *1.0* – however, the differences are smaller than for the *columns*. Indeed, the DSM local-plate and distortional strength estimates are now, *28%* and *15%* higher than the "exact" values (again in average) – the σ_{nl}/σ_u and σ_{nd}/σ_u standard deviations are also less pronounced: *0.24* and *0.17*. Even then, σ_{nl} and σ_{nd} overestimate σ_u by a fair amount in beams affected by local-plate/distortional mode interaction.

Table 2B. Comparison between the "exact" ultimate strengths and their DSM estimates (σ_{nl}, σ_{nd} and σ_{nld}) for *30* (out of *90*) beams

			FEA			DSM					
	b_w	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_f=50, b_s=10, t=0.8, L=450mm	60	250	241	256	203	210	1.03	197	0.97	178	0.86
	60	350	241	256	222	262	1.18	243	1.09	206	0.91
	60	450	241	256	228	309	1.36	283	1.24	228	0.99
	70	250	239	242	200	209	1.05	193	0.96	176	0.86
	70	350	239	242	215	262	1.22	238	1.11	203	0.92
	70	450	239	242	223	309	1.38	277	1.24	224	0.99
	80	250	237	231	197	209	1.06	189	0.96	173	0.86
	80	350	237	231	223	261	1.17	233	1.05	199	0.87
	80	450	237	231	229	308	1.34	271	1.19	221	0.94
	90	250	234	220	193	208	1.08	186	0.96	171	0.86
	90	350	234	220	210	260	1.24	229	1.09	196	0.91
	90	450	234	220	215	307	1.43	266	1.24	217	0.98
	100	250	231	211	190	207	1.09	183	0.96	168	0.86
	100	350	231	211	203	259	1.27	225	1.11	193	0.92
	100	450	231	211	210	305	1.45	262	1.25	213	0.99
	b_f	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=180, b_s=15, t=1.1, L=750mm	60	250	210	252	207	201	0.97	195	0.94	170	0.82
	60	350	210	252	240	251	1.04	241	1.01	196	0.82
	60	450	210	252	252	295	1.17	281	1.12	217	0.86
	65	250	204	221	196	198	1.01	186	0.95	163	0.82
	65	350	204	221	219	248	1.13	229	1.05	187	0.85
	65	450	204	221	223	292	1.31	267	1.20	207	0.92
	70	250	194	196	184	195	1.06	178	0.97	156	0.83
	70	350	194	196	195	244	1.25	219	1.12	179	0.90
	70	450	194	196	200	287	1.44	254	1.27	198	0.97
	75	250	182	176	173	191	1.10	171	0.99	148	0.84
	75	350	182	176	180	238	1.32	210	1.16	170	0.92
	75	450	182	176	185	281	1.52	243	1.31	187	0.99
	80	250	167	160	154	185	1.20	165	1.07	141	0.89
	80	350	167	160	160	231	1.44	201	1.26	161	0.98
	80	450	167	160	164	272	1.66	233	1.42	177	1.06

Finally, the variation of σ_{nd}/σ_u with the distortional slenderness λ_d is shown in figures 2(a) (columns) and 2(b) (beams). It is clear that the DSM distortional failure expressions yield fairly accurate and mostly safe ultimate strength estimates for the stockier *columns* and *beams* ($\lambda_d \leq 1.2$). However, these same expressions perform poorly for moderate-to-slender columns and beams ($\lambda_d \geq 1.2$), *i.e.*, their predictions are inaccurate and mostly unsafe – moreover, the error grows with λ_d. This means that the presence of local-plate buckling

Table 2C. Comparison between the "exact" ultimate strengths and their DSM estimates (σ_{nl}, σ_{nd} and σ_{nld}) for *30* (out of *90*) beams and overall results

			FEA			DSM					
	b_s	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_w=400, b_f=150, t=2.0, L=1400mm	22	250	131	115	126	171	1.36	144	1.14	119	0.91
	22	350	131	115	130	212	1.63	175	1.35	135	1.01
	22	450	131	115	133	250	1.88	202	1.52	148	1.08
	24	250	132	123	131	171	1.30	148	1.13	121	0.90
	24	350	132	123	134	213	1.59	180	1.35	138	1.00
	24	450	132	123	138	250	1.81	208	1.51	152	1.07
	26	250	132	132	137	171	1.25	152	1.11	123	0.88
	26	350	132	132	140	213	1.52	186	1.33	141	0.99
	26	450	132	132	143	250	1.75	214	1.50	155	1.07
	28	250	132	142	142	171	1.21	157	1.11	126	0.88
	28	350	132	142	146	213	1.46	192	1.31	144	0.98
	28	450	132	142	149	250	1.68	221	1.49	158	1.05
	30	250	132	163	147	171	1.16	166	1.13	131	0.89
	30	350	132	163	153	213	1.39	203	1.33	149	0.98
	30	450	132	163	156	250	1.60	235	1.51	164	1.06
	b_w	f_y	σ_{crl}	σ_{crd}	σ_u	σ_{nl}	σ_{nl}/σ_u	σ_{nd}	σ_{nd}/σ_u	σ_{nld}	σ_{nld}/σ_u
b_f=100, b_s=12, t=1.4, L=750mm	350	250	91.1	83.9	104	150	1.44	126	1.22	96	0.90
	350	350	91.1	83.9	113	186	1.65	153	1.35	109	0.94
	350	450	91.1	83.9	121	219	1.81	176	1.45	120	0.97
	370	250	82.7	80.0	102	145	1.42	124	1.21	92	0.88
	370	350	82.7	80.0	111	180	1.62	150	1.35	104	0.92
	370	450	82.7	80.0	120	211	1.76	172	1.43	114	0.94
	390	250	75.3	76.5	101	140	1.39	121	1.20	88	0.86
	390	350	75.3	76.5	110	174	1.58	147	1.33	99	0.90
	390	450	75.3	76.5	118	204	1.73	169	1.43	109	0.92
	410	250	68.1	72.3	99.1	135	1.37	119	1.20	84	0.84
	410	350	68.1	72.3	109	168	1.54	143	1.31	95	0.86
	410	450	68.1	72.3	117	197	1.68	164	1.41	103	0.88
	430	250	61.9	68.3	97.5	131	1.34	116	1.19	80	0.82
	430	350	61.9	68.3	107	162	1.51	140	1.30	90	0.84
	430	450	61.9	68.3	115	190	1.65	160	1.39	98	0.86
						Av.	**1.28**	**Av.**	**1.15**	**Av.**	**0.95**
						Sd.	**0.241**	**Sd.**	**0.167**	**Sd.**	**0.069**

effects leads to a substantial erosion of the column or beam ultimate strength associated with the distortional failure. In addition, this erosion grows as the yield-to-critical distortional stress ratio f_y/σ_{crd} increases. Therefore, the influence of the local-plate/distortional mode interaction phenomenon on the ultimate strength of columns or beams (distortional failure) must be taken into account whenever their slenderness value λ_d is moderate-to-high.

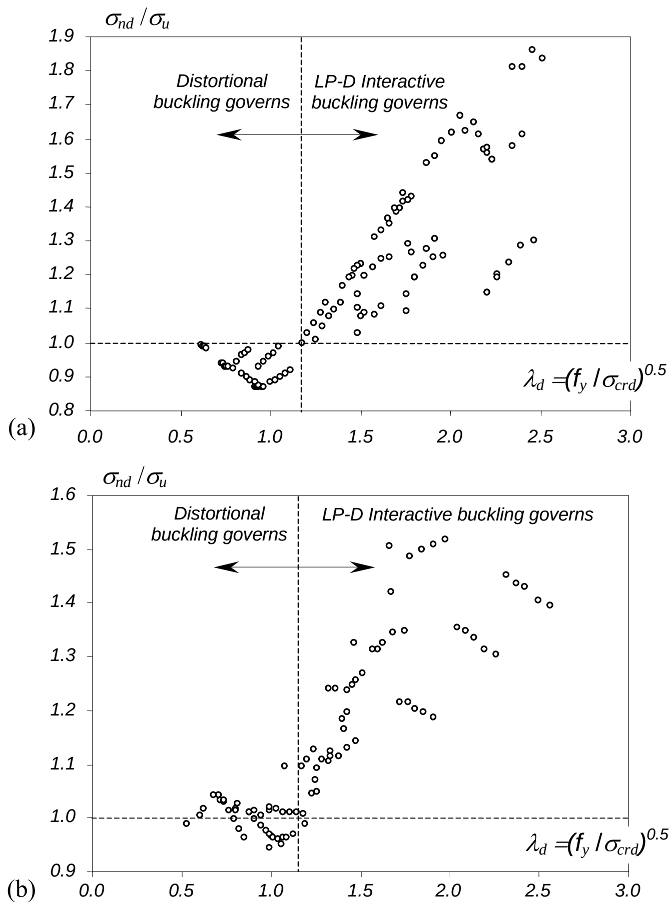

Fig. 2: Variation of σ_{nd}/σ_u with λ_d for (a) columns and (b) beams

DSM for Local-Plate/Distortional Interaction

Following a strategy similar to the one adopted to develop a DSM approach to estimate the ultimate strength of columns and beams exhibiting a local-plate/global interactive buckling behavior, it becomes possible to propose expressions that are applicable to columns and beams experiencing local-plate/distortional mode interaction effects. For the columns, this can be done by replacing either (i) P_y by P_{nd} in eqs. (1) or (ii) P_y by P_{nl} in eqs. (2) – P_{nd} and P_{nl} are the distortional and local-plate buckling strengths given by eqs. (1) and (2). Then, one obtains ultimate load estimates P_{nld} and P_{ndl}, respectively. Yang & Hancock (2004) have recently adopted the first approach, which is schematically presented in the flowchart of figure 3(a) – the "role" of the overall strength P_{ne} is now played by the distortional strength P_{nd}. Finally, note that the approach just outlined involves the knowledge of *accurate* local-plate and distortional buckling loads (P_{crl}, P_{crd}), which can be readily determined through finite element, finite strip or generalised beam theory (GBT) analyses. The same methodology can also be applied to the beams, as illustrated in the flowchart of figure 3(b) – one then obtains a M_{nld} value, which estimates the corresponding beam ultimate strength.

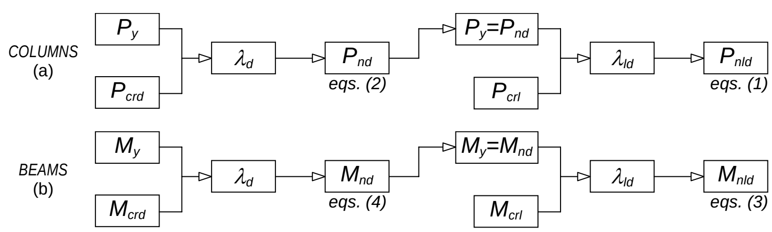

Fig. 3: Flowcharts concerning the application of the DSM to (a) columns (P_{nld}) and (b) beams (M_{nld}) under local-plate/distortional mode interaction

After comparing the ultimate strength estimates provided by their DSM approach with the results of a series of experimental tests involving lipped channel columns with "v-shaped" web and flange intermediate stiffeners (Yang 2004), which provided clear evidence of an adverse local-plate/distortional interaction, Yang & Hancock (2004) concluded that (i) the above estimates were safe and reasonably accurate (differences in the *10-20%* range), and also that (ii) further investigation was required concerning the design of columns with nearly coincident local-plate and distortional buckling stresses. On the other hand, Silvestre *et al.* (2005), in the context of simply supported "plain" lipped channel columns experiencing local-plate/distortional mode interaction, compared the two aforementioned DSM approaches (P_{nld} and P_{ndl} – *LD* and *DL* approaches) and concluded that they lead to very similar ultimate strength estimates. In view of these facts, it was decided to adopt the "*LD* approach" and employ it to estimate the ultimate strength of the columns and beams addressed in this work.

Assessment of DSM Estimates for LP/D Interaction

Besides the predictions yielded by the individual local-plate and distortional DSM failure expressions, tables 1A-C and 2A-C also include DSM σ_{nld} estimates. Their observation leads to the following remarks:

(i) Although the column σ_{nld} estimates are reasonably accurate in average (mean of σ_{nld}/σ_u equal to *0.97*), there are well scattered: the σ_{nld}/σ_u standard deviation is *0.13*. Among the whole set of σ_{nld} estimates, *1* is exact, *38* are safe and accurate ($\sigma_{nld}/\sigma_u \geq 0.9$), *36* are excessively safe ($0.79 \leq \sigma_{nld}/\sigma_u < 0.90$), *16* are slightly unsafe ($\sigma_{nld}/\sigma_u \leq 1.10$) and *17* are very unsafe ($1.10 < \sigma_{nld}/\sigma_u \leq 1.36$).

(ii) The beam σ_{nld} estimates are also reasonably accurate in average (mean of σ_{nld}/σ_u equal to *0.95*). However, unlike in the columns, the scatter is now quite low: σ_{nld}/σ_u standard deviation of *0.069*. Out of the whole set of σ_{nld} estimates, *3* are exact, *44* are safe and accurate ($\sigma_{nld}/\sigma_u \geq 0.9$), *28* are too safe ($0.82 \leq \sigma_{nld}/\sigma_u < 0.90$) and *15* are accurate but slightly unsafe ($\sigma_{nld}/\sigma_u \leq 1.10$).

The variation of the stress ratios σ_u/f_y and σ_{nld}/f_y with the *distortional slenderness* $\lambda_d = (f_y/\sigma_{crd})^{0.5}$ are shown in figures 4(a) (columns) and 4(b) (beams). Also included are the "Winter-type" curves defined by eqs. (1)-(2) (columns) and (3)-(4) (beams), which provide the DSM *local-plate* and *distortional* ultimate strength estimates. From the joint observation of all these results, the following comments can be drawn:

(i) The proposed DSM predictions (black dots) always (i_1) lie well below both the local-plate and distortional curves, for the *slender* members ($\lambda_d > 1.2$), and (i_2) are located near the distortional curve, for the *stockier* members ($\lambda_d < 1.2$). This means that, at least for the critical stress ratio range considered ($0.90 \leq \sigma_{crd}/\sigma_{crl} \leq 1.10$), the local-plate/distortional mode interaction always causes a substantial strength erosion in the *slender* members (w.r.t. the individual local-plate and distortional values).

(ii) Regardless of the member distortional slenderness values, the black dots (ii_1) always remain quite "aligned" and (ii_2) lie in a fairly close vicinity of the "exact" ultimate strength values (white dots), in spite of their "vertical dispersion" – they lie mostly below (particularly in the beams).

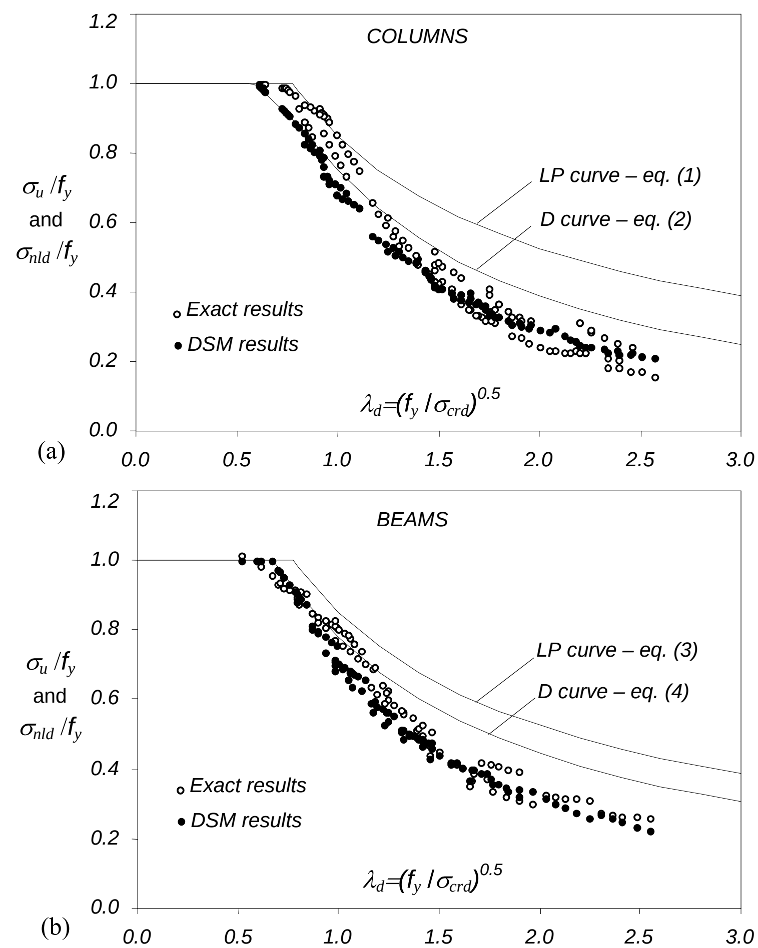

Fig. 4: Variation of σ_{nld}/f_y and σ_u/f_y with λ_d for (a) columns and (b) beams

Design Recommendations

All the members analyzed (a total of *198*) display distortional slenderness values falling inside the range for which the individual local-plate and distortional DSM curves were experimentally calibrated. If the column or beam

σ_{crl} and σ_{crd} values are less than *15%* apart, it seems fair to say that:

(i) In members with a low slenderness ($\lambda_d \leq 1.2$), the σ_{nd} values accurately predict the ultimate strength σ_u under local-plate/distortional mode interaction – thus, the current DSM provisions for distortional failure can be satisfactorily employed.

(ii) In members with a moderate-to-high slenderness ($\lambda_d \geq 1.2$), the current DSM provisions yield unsatisfactory results, while the σ_{nld} approach yields mostly accurate predictions of the ultimate strength σ_u under local-plate/distortional mode interaction – however, for wide-flange columns with high yield stresses, σ_{nld} may provide unsafe ultimate strength estimates (see Silvestre *et al.* 2005 for more details).

(iii) Regardless of the member slenderness, the σ_{nld} values provide fairly accurate predictions of the ultimate strength of member prone to local-plate/distortional mode interaction (with the exception mentioned in the previous item) – such an achievement cannot be reached through the use of the current DSM (local-plate and distortional) provisions.

Conclusion

Results of an ongoing investigation on the use of the Direct Strength Method to estimate the ultimate strength of lipped channel columns and beams affected by local-plate/distortional interaction were presented and discussed. On the basis of the results of a FEM-based parametric study involving *108* columns and *90* beams, it was possible (i) to obtain numerical evidence of the severe member strength erosion caused by the local-plate/distortional interactive and (ii) to reveal the inability of the DSM individual (local-plate and distortional) expressions to predict such erosion in arbitrary lipped channel columns and beams. However, it was shown that a DSM approach based on those individual expressions (recently proposed by Yang & Hancock 2004 for columns) predicts quite well the strength reduction caused to lipped channel columns and beams by local-plate/distortional mode interaction – even so, it was also possible to identify a number of features that must be included in a more elaborate DSM approach, specifically developed to take into account this type of interactive behavior. Finally, the paper closed with some design recommendations about the application of the DSM approach dealt with here to lipped channel members exhibiting nearly coincident local-plate and distortional buckling stresses.

References

AISI (American Iron and Steel Institute) (2004), *Appendix I of the North American Specification (NAS) for the Design of Cold-Formed Steel Structural Members: Design of Cold-Formed Steel Structural Members with the Direct Strength Method*, Washington DC.

Dinis P.B. and Camotim D. (2004). "Local-plate and distortional post-buckling behavior of cold-formed steel columns: elastic and elastic-plastic FEM analysis", *Proceedings of SSRC Annual Stability Conference,* (Long Beach, 24-27/3), 475-498.

Dinis P.B. and Camotim D. (2006). "On the use of shell finite element analysis to assess the local buckling and post-buckling behaviour of cold-formed steel thin-walled members", *Proceedings of the III European Conference on Computational Mechanics: Solids, Structures and Coupled Problems in Engineering* (ECCM 2006 – Lisboa, 5-9/6) (in press)

Dinis P.B., Silvestre N. and Camotim D. (2005a). "Analysis of the local-plate/distortional mode interaction in cold-formed steel lipped channel columns", *Proceedings of Tenth International Conference on Civil, Structural and Environmental Engineering Computing* (Rome, 30-8/1-9), B. Topping (ed.), Civil-Comp Press, 371-372. (full paper in CD-ROM Proceedings)

Dinis P.B., Camotim D. and Silvestre N. (2005b). "FEM-based analysis of the local-plate/distortional mode interaction in cold-formed steel lipped channel columns", *submitted for publication*.

Duong H.M. and Hancock G.J. (2004). "Recent developments in the direct strength design of thin-walled members", *Thin-Walled Structures: Recent Advances and Future Trends in Thin-Walled Structures Technology*, J. Loughlan (ed.), Canopus Publishing, 43-62.

Hancock G.J., Murray T.M. and Ellifritt D.S. (2001). *Cold-Formed Steel Structures to the AISI Specification,* Marcel Dekker, New York.

HKS (Hibbit, Karlsson and Sorensen Inc.) (2002). *ABAQUS Standard* (version 6.3-1).

Kwon Y.B., Kim N.G. and Kim B.S. (2005). "A study on the direct strength method for compression members undergoing mixed mode buckling", *Proceedings of Third International Symposium on Steel Structures* (Seoul, 10-11/3), 108-119.

Martins A. (2006), *Local-Plate/Distortional Buckling Mode interaction in Cold-Formed Steel Lipped Channel Beams*, M.A.Sc. Thesis, IST, Technical University of Lisbon, Portugal. (in progress) (Portuguese)

Rasmussen K.J. and Hossain M.S. (2004). "Design of slender angle section beam-columns by the direct strength method", *Proceedings of 4th International Conference on Coupled Instabilities in Metal Structures* (CIMS'04 – Rome, 27-29/9), 499-509.

SA-SNZ (Standards of Australia and Standards of New Zealand) (2005), *Australian/New Zealand Standard on Cold-Formed Steel Structures* – AS/NZS 4600 (second edition), Sydney-Wellington.

Schafer B.W. (2002). "Progress on the direct strength method", *Proceedings of 16th International Specialty Conference on Cold-Formed Steel Structures* (Orlando, 17-18/10), 647-662.

Schafer B.W. (2003a). "Cold-formed steel design by the direct strength method: bye-bye effective width", *Proceedings of SSRC Annual Technical Session & Meeting* (Baltimore, 2-5/4), 357-377.

Schafer B.W. (2003b). "Advances in the direct strength design of thin-walled members", *Advances in Structures* (ASSCCA'03 – Sydney, 23-25/6), Hancock *et al.* (eds.), Balkema, Lisse, 333-339.

Schafer B.W. (2005), *Direct Strength Method Design Guide*, American Iron and Steel Institute (AISI), Washington DC.

Schafer B.W. and Peköz T. (1999). "Laterally braced cold-formed steel flexural members with edge stiffened flanges", *Journal of Structural Engineering* (ASCE), **125**(2), 118-127.

Silvestre N., Camotim D. and Dinis P.B. (2005). "On the use of the direct strength method to design lipped channel columns affected by local-plate/distortional mode interaction", *Proceedings of the 4th European Conference on Steel and Composite Structures* (EUROSTEEL 2005 – Maastricht, 8-10/6), 125-133.

Yang D. (2004). *Compression Stability of High Strength Steel Sections with Low Strain Hardening*, Ph.D. Thesis, School of Civil and Mining Engineering, University of Sydney.

Yang D. and Hancock G.J. (2004). "Compression tests of high strength steel channel columns with interaction between local and distortional buckling", *Journal of Structural Engineering* (ASCE), **130**(12), 1954-1963.