Cold-Formed Steel Channel Sections with Web Stiffeners Subjected to Local and Distortional Buckling — Part II: Parametric Study and Design Rule

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Twenty-Second International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, USA, November 5 & 6, 2014

Cold-formed Steel Channel Sections with Web Stiffeners subjected to Local and Distortional Buckling — Part II: Parametric Study and Design Rule

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Abstract

A parametric study of cold-formed steel channel sections with web stiffeners subjected to bending was performed using finite element analysis (FEA). An accurate finite element model was used for the parametric study. The parametric study included 75 beams of plain and lipped channel sections with web stiffeners. The beams were simply supported and subjected to four-point bending. The strengths and failure modes of specimens obtained from experimental and FEA results were compared with design strengths predicted using the direct strength method (DSM) specified in the North American Specification for cold-formed steel structures. The comparison shows that the design strengths predicted by the current DSM are conservative for both local buckling and distortional buckling in this study. Hence, the DSM is modified to cover the new stiffened channel sections investigated in this study. A reliability analysis was also performed to assess the current and modified DSM.

Introduction

The advantages of using cold-formed steel sections are high strength-to-weight ratio, flexibility in fabricating different cross-section shapes, easy for construction and so on. Local buckling and distortional buckling are usually the governing failure modes for cold-formed steel sections, such as thin-walled plain channel and lipped channel sections. In plate mechanics, the edge

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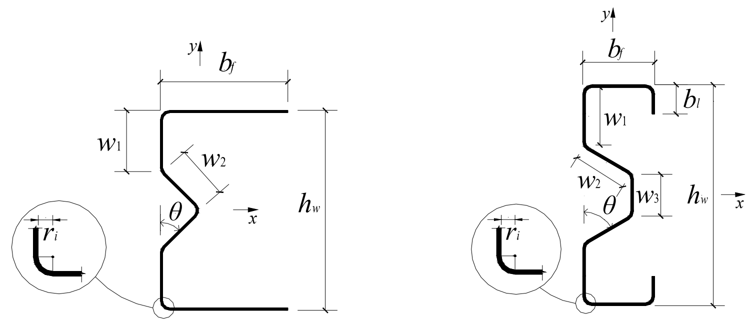
stiffeners, such as lips in channel sections, and intermediate stiffeners in the web can enhance the strength of sections by acting as the out-of-plane supports to the flat plate elements of sections. Thus, the stiffeners improve the efficiency of the use of material.

Design rules of cold-formed steel structural members can be found in the international specifications, such as the European Code (EC3, 2006), North American Specification (NAS, 2012) and Australian/New Zealand Standard (AS/NZS, 2005). Two main design methods, namely the effective width method (EWM) and the direct strength method (DSM), are used to calculate members failed by local buckling and distortional buckling. However, when sections were stiffened by edge and intermediate stiffeners for optimized section shapes, the computation of effective width for each plate element could be quite tedious that involves iteration processes and the EWM becomes much more complicated compared to the DSM. Hence, the DSM was recommended for design of cold-formed steel members with complex stiffeners (Schafer et al., 2006). On the other hand, the DSM in current specifications is a semi-empirical approach (Schafer, 2008), which was calibrated to cover only the pre-qualified sections specified in NAS (2012).

A test program of cold-formed steel channel sections with web stiffeners subjected to bending has been presented by Wang & Young (2014) in the companion paper. An accurate finite element model has also been developed and verified against the tests by Wang & Young (2014). The purpose of this paper is firstly to investigate the behaviour and design of stiffened plain channel (Fig. 1(a)) and lipped channel (Fig. 1(b)) sections with various geometries by performing a parametric study using finite element analysis. Secondly, the appropriateness of DSM in current specifications was evaluated for the stiffened sections in this study based on the experimental and numerical results. Finally, modified DSM is proposed for cold-formed steel stiffened channel sections beams subjected to local and distortional buckling.

Summary of Test Program

The test program presented by Wang & Young (2014) in the companion paper provided experimental moment capacities and failure modes of cold-formed steel channel sections with web stiffeners subjected to bending about the major *x*-axis. The test program included 12 plain channels with web stiffeners and 14 lipped channels with web stiffeners. Two identical stiffened channels were tested at the same time in order to avoid out-of-plane bending. Therefore, a total of 13 simply supported beams were tested under both four-point bending and three-point. The test specimens were cut into a specified length of 1400 mm for all the channels. The beam tests were conducted using displacement control. The material properties of the test specimens were obtained by carrying out tensile coupon tests. The details of the test program have been reported in Wang & Young (2014).



(a) PWS-section (b) LWS-section

Figure 1: Definition of symbols

Finite Element Modelling

The finite element package ABAQUS (2011) was used to develop a finite element model (FEM) and perform nonlinear analysis of the test beams with stiffened channel sections subjected to four-point bending. Only one channel of the beam was modelled due to symmetry. The material model, boundary condition and loading condition as well as the element type and mesh are detailed in Wang & Young (2014). The developed FEM was verified against the experimental results. The FEM closely predicted the behaviour of cold-formed steel channel sections with web stiffeners subjected to local and distortional buckling.

Parametric Study

It has been shown that the finite element model well predicted the moment capacities and failure modes of the test beams in Wang & Young (2014). Thus, the verified model was used for an extensive parametric study of 75 beams with stiffened channel sections subjected to four-point bending. These sections were symmetric about the axis of bending.

The stiffened channel sections in the parametric study were designed using the finite strip analysis (Papangelis & Hancock, 1995), which can predict the elastic buckling stresses as well as the failure modes. A total of 26 sections was investigated. Each section has two to three thicknesses that ranged from 0.48 to 3.6 mm in order to cover a wide range of section slenderness. The flange slenderness (b_f/t), overall web depth-to-thickness ratio (h_w/t), and the geometry of stiffeners in the channel sections were investigated. The beam length was 1400 mm with a constant moment span of 600 mm for all specimens, which allows local buckling and distortional buckling to be the dominant failure modes. The specimens were labelled such that the section shape, plate thickness, the characteristic of stiffeners as well as the overall section sizes could be identified, as shown in Fig. 2. It should be noted that three of the PWS-section specimens have two symmetric stiffeners located at the points one-third of the web, and a symbol "*" was included in the labels. The length of web element (w_3) of LWS-section specimens is identical to the test specimens, except for the nine specimens with symbol "^" in the labels. However, the length of inclined web element (w_2) is only half of the test specimens, so that the length of web element (w_3) could be determined for these nine specimens.

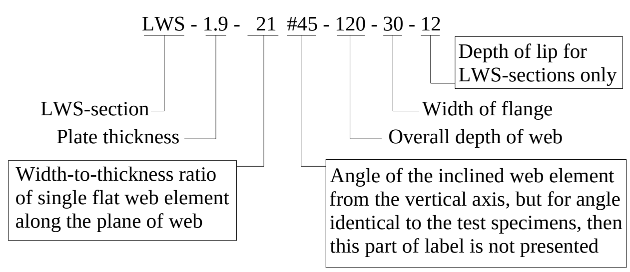


Figure 2: Label of specimens for parametric study

The moment capacities per channel (M_{FEA}) and the corresponding failure modes obtained from the parametric study are summarized in Table 1 for specimens failed by local buckling, and Table 2 for specimens failed by distortional buckling.

Design Approach

The stiffened channel sections investigated in this study are not within the geometric limitations prescribed in NAS (2012) and AS/NZS (2005) when the DSM for beams is used. Hence, the appropriateness of the DSM on the cold-formed steel stiffened channel sections subjected to bending was evaluated.

Direct Strength Method for Cold-formed Steel Beams

In this study, no lateral-torsional buckling occurred to the specimens in the tests and parametric study, so the specimens could be regarded as fully braced beams. Hence, the nominal flexural strength (M_{ne}) for lateral-torsional buckling is taken as the yield moment (M_y) for fully braced beams (NAS, 2012). The current DSM for beams that considered inelastic reserve capacities for local buckling and distortional buckling in NAS (2012) are summarized as follows.

The nominal flexural strength (M_{DSM}), is the minimum of nominal flexural strength for local buckling (M_{nl}) and nominal flexural strength for distortional buckling (M_{nd}), as shown in Eq. (1):

$$M_{DSM} = \min(M_{nl}, M_{nd}) \tag{1}$$

The nominal flexural strength for local buckling (M_{nl}) of sections symmetric about the axis of bending is calculated in accordance with the following:

For $\lambda_l \le 0.776$, $$M_{nl} = M_y + (1 - 1/C_{yl}^2)(M_p - M_y) \tag{2}$$

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

where $\lambda_l = \sqrt{M_y / M_{crl}}$; $C_{yl} = \sqrt{0.776 / \lambda_l} \le 3$; $M_y = S_f f_y$; $M_p = Z_f f_y$; S_f = gross section modulus referenced to the extreme fiber at first yield; Z_f = Plastic section modulus; f_y = yield stress which is the 0.2% proof stress ($\sigma_{0.2}$) obtained from tensile coupon tests in this study; M_{crl} = critical elastic local buckling moment ($M_{crl} = S_f \sigma_{crl}$).

The nominal flexural strength for distortional buckling (M_{nd}) of sections symmetric about the axis of bending is calculated in accordance with the following:

For $\lambda_d \le 0.673$, $$M_{nd} = M_y + (1 - 1/C_{yd}^2)(M_p - M_y) \tag{4}$$

For $\lambda_d > 0.673$, $$M_{nd} = \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \tag{5}$$

where $\lambda_d = \sqrt{M_y / M_{crd}}$; $C_{yd} = \sqrt{0.673/\lambda_d} \le 3$; M_{crd} = critical elastic distortional buckling moment ($M_{crd} = S_f \sigma_{crd}$).

The elastic local buckling stress (σ_{crl}) and elastic distortional buckling stress (σ_{crd}) were obtained from the finite strip analysis (Papangelis & Hancock, 1995).

Reliability Analysis

Reliability analysis was performed in order to evaluate the appropriateness of the current DSM for the cold-formed steel stiffened channel sections subjected to bending in this study. The target reliability index for structural members for LRFD is 2.5 according to the North American Specification (Section F1.1 (c) of NAS (2012)). The resistance factor (ϕ_b) of 0.8 was used in the analysis as specified in Section A1.2 (c) of NAS (2012) and Section 1.6.3 (c) of AS/NZS (2005). In addition, the resistance factor (ϕ_b) of 0.9 was also used in the analysis.

Specimens	Tests or FEA (per channel)		DSM predictions			Comparison	
	M_{EXP} / M_{FEA} (kNmm)	Failure mode	λ_l	M_{DSM} (kNmm)	M_{DSM*} (kNmm)	$\frac{M_{EXP}}{M_{DSM}}$ / $\frac{M_{FEA}}{M_{DSM}}$	$\frac{M_{EXP}}{M_{DSM*}}$ / $\frac{M_{FEA}}{M_{DSM*}}$
PWS-0.48-B4	591	L+F	6.414	375	638	1.57	0.93
PWS-0.48-B4R	678	L+F	6.414	375	638	1.81	1.06
PWS-1.0-B4	1809	L+F	2.888	1351	1876	1.34	0.96
PWS-1.0-B4R	1929	L+F	2.888	1351	1876	1.43	1.03
PWS-3.6-10-120-30	14706	F	0.483	13822	13888	1.06	1.06
PWS-3.6-13-120-30	14028	F	0.493	13574	13734	1.03	1.02
PWS-1.9-19-120-30	6690	L+F	0.891	6160	6683	1.09	1.00
PWS-1.9-25-120-30	6855	L+F	0.912	6015	6550	1.14	1.05
PWS-2.4-19-150-52	14785	L+F	1.208	12367	14156	1.20	1.04
PWS-2.4-13*-150-52	15345	L+F	1.209	12551	14369	1.22	1.07
PWS-2.4-19#30-150-52	14633	L+F	1.217	12247	14037	1.20	1.04
PWS-2.4-25-150-52	13890	L+F	1.232	12111	13915	1.15	1.00
PWS-1.9-19-120-52	7801	L+F	1.503	6279	7509	1.24	1.04
PWS-1.9-13-82.5-52	4666	L+F	1.503	3922	4691	1.19	1.00
PWS-1.9-9-82.5-52	4759	L+F	1.512	3956	4737	1.20	1.01
PWS-1.9-9#30-82.5-52	4553	L+F	1.530	3881	4659	1.17	0.98
PWS-1.9-17-82.5-52	4005	L+F	1.531	3857	4631	1.04	0.87

PWS-1.9-25-120-52	7333	L+F	1.535	6149	7386	1.19	0.99
PWS-1.9-63-120-52	6142	L+F	1.583	5893	7125	1.04	0.86
PWS-1.0-36-120-30	2955	L+F	1.673	2111	2583	1.40	1.14
PWS-1.0-48-120-30	2498	L+F	1.709	2060	2533	1.21	0.99
PWS-1.5-11-82.5-52	3087	L+F	1.876	2680	3363	1.15	0.92
PWS-1.5-17-82.5-52	3413	L+F	1.879	2642	3317	1.29	1.03
PWS-1.5-20*-150-52	9314	L+F	1.889	5736	7208	1.62	1.29
PWS-1.5-11#30-82.5-52	3335	L+F	1.911	2615	3294	1.28	1.01
PWS-1.5-30-150-52	8627	L+F	1.920	5581	7039	1.55	1.23
PWS-1.5-22-82.5-52	2893	L+F	1.921	2588	3265	1.12	0.89
PWS-1.5-30#30-150-52	8254	L+F	1.924	5543	6995	1.49	1.18
PWS-1.5-40-150-52	6724	L+F	1.956	5468	6925	1.23	0.97
PWS-1.0-17-82.5-52	2026	L+F	2.736	1355	1857	1.50	1.09
PWS-1.0-17#30-82.5-52	1859	L+F	2.787	1322	1819	1.41	1.02
PWS-1.0-30*-150-52	5282	L+F	2.799	2865	3948	1.84	1.34
PWS-1.0-36-120-52	3168	L+F	2.826	2089	2886	1.52	1.10
PWS-1.0-33-82.5-52	1969	L+F	2.838	1296	1792	1.52	1.10
PWS-1.0-45-150-52	4433	L+F	2.870	2769	3838	1.60	1.16
PWS-1.0-45#30-150-52	4534	L+F	2.871	2756	3821	1.65	1.19
PWS-1.0-48-120-52	2749	L+F	2.877	2049	2842	1.34	0.97
PWS-1.0-60-150-52	3805	L+F	2.922	2713	3777	1.40	1.01
PWS-1.0-120-120-52	2561	L+F	2.996	1948	2729	1.32	0.94
PWS-0.6-60-120-52	1447	L+F	4.687	856	1342	1.69	1.08
PWS-0.6-80-120-52	1417	L+F	4.763	840	1323	1.69	1.07
					Mean (P_m)	1.34	1.04
					COV (V_P)	0.165	0.099
					ϕ_b =0.8, Reliability index (β_1)	3.53	3.01
					ϕ_b =0.8, Reliability index (β_2)	3.36	2.82
					ϕ_b =0.9, Reliability index (β_1)	3.13	2.55
					ϕ_b =0.9, Reliability index (β_2)	2.96	2.36

L=Local buckling; F=Flexural buckling

Table 1: Comparison of moment capacities obtained from test and FEA results with DSM predictions for sections subjected to local buckling

The load combinations of 1.2 DL + 1.6 LL as specified in the American Society of Civil Engineers Standard (ASCE, 2006), and 1.25 DL + 1.5 LL as specified in the Australian Standard (AS/NZS, 2002) were adopted in the calculation, where DL is the dead load and LL is the live load. The live load to dead load ratio of 1/5 was used, which is consistent with Eq. (F1.1-2) of NAS (2012). Other statistical parameters were obtained from Table F1 of NAS (2012) for bending strength of beams, where $M_m = 1.10$, $F_m = 1.00$, $V_M = 0.10$ and $V_F = 0.05$ are the mean values and coefficients of variation of material factor and fabrication factor, respectively. The statistical parameters P_m and V_P are the mean value and coefficient of variation of experimental/FEA-to-predicted moment ratio, respectively. A correction factor C_P was also used in the reliability calculation to account for the influence of limited number of data samples. The reliability index (β_1) was calculated using the load combination of 1.2 DL + 1.6 LL, while reliability index (β_2) was calculated using the load combination of 1.25 DL + 1.5 LL, as shown in Tables 1 and 2.

	Tests or FEA (per channel)		DSM predictions			Comparison	
Specimens	M_{EXP} / M_{FEA} (kNmm)	Failure mode	λ_d	M_{DSM} (kNmm)	M_{DSM*} (kNmm)	$\frac{M_{EXP}}{M_{DSM}}$ / $\frac{M_{FEA}}{M_{DSM}}$	$\frac{M_{EXP}}{M_{DSM*}}$ / $\frac{M_{FEA}}{M_{DSM*}}$
LWS-0.48-B4	1029	L+D+F	1.408	973	1021	1.06	1.01
LWS-1.0-B4	2985	D+F	0.884	2706	3098	1.10	0.96
LWS-1.2-B4	3807	D+F	0.819	3476	3926	1.10	0.97
LWS-3.6-7-94-30-18	13171	D+F	0.416	12294	12457	1.07	1.06
LWS-3.6-15-150-30-12	23654	F	0.436	21969	22243	1.08	1.06
LWS-3.6-7-94-30-6	11370	D+F	0.470	10573	10823	1.08	1.05
LWS-2.4-11-94-30-18	8638	D+F	0.519	7936	8126	1.09	1.06
LWS-2.4-11-94-30-12	8027	D+F	0.545	7453	7672	1.08	1.05
LWS-2.4-11^-94-30-12	7892	D+F	0.548	7356	7602	1.07	1.04
LWS-1.9-21-120-30-12	8774	D+F	0.621	7981	8271	1.10	1.06
LWS-1.9-28-150-30-12	11960	F	0.629	10862	11259	1.10	1.06
LWS-1.2-22-94-20-12	3187	D+F	0.632	2944	3047	1.08	1.05
LWS-1.9-21#45-120-30-12	8599	F	0.648	7840	8158	1.10	1.05
LWS-3.6-15-150-60-12	31805	D+F	0.746	28381	30802	1.12	1.03
LWS-1.2-22-94-30-18	3855	D+F	0.749	3550	3859	1.09	1.00
LWS-1.2-33-120-30-12	5001	D+F	0.797	4499	5021	1.11	1.00
LWS-1.2-28^-94-30-12	3686	D+F	0.803	3178	3558	1.16	1.04
LWS-1.2-22^-94-30-12	3702	D+F	0.807	3196	3587	1.16	1.03
LWS-1.2-45-150-30-12	6900	F	0.809	6079	6784	1.14	1.02
LWS-1.2-16^-94-30-12	3770	D+F	0.811	3228	3632	1.17	1.04
LWS-1.2-22#30-94-30-12	3569	D+F	0.828	3110	3531	1.15	1.01
LWS-1.2-33#45-120-30-12	5102	D+F	0.836	4333	4939	1.18	1.03
LWS-0.6-43-94-20-12	1361	L+D+F	0.906	1209	1377	1.13	0.99
LWS-1.2-22-94-30-6	3207	D+F	0.911	2753	3131	1.17	1.02
LWS-0.48-54-94-20-12	1015	D+F	1.016	893	993	1.14	1.02
LWS-0.75-35-94-30-12	2004	D+F	1.019	1720	1911	1.17	1.05
LWS-0.75-35^-94-30-12	2086	D+F	1.037	1683	1864	1.24	1.12

LWS-1.2-22-94-45-12	4083	D+F	1.063	3429	3779	1.19	1.08
LWS-0.75-35#30-94-30-12	1986	D+F	1.067	1630	1795	1.22	1.11
LWS-0.6-65-120-30-12	1690	L+D+F	1.146	1745	1897	0.97	0.89
LWS-0.6-55^-94-30-12	1430	D+F	1.161	1227	1330	1.17	1.08
LWS-0.6-32^-94-30-12	1482	D+F	1.172	1245	1348	1.19	1.10
LWS-0.6-65#45-120-30-12	1656	L+D+F	1.208	1664	1791	1.00	0.92
LWS-0.48-69^-94-30-12	852	L+D+F	1.303	896	953	0.95	0.89
LWS-0.48-40^-94-30-12	944	L+D+F	1.316	910	965	1.04	0.98
LWS-0.6-43-94-30-6	1224	D+F	1.329	1039	1100	1.18	1.11
LWS-0.48-54#30-94-30-12	865	L+D+F	1.350	870	919	1.00	0.94
LWS-0.75-35-94-45-12	1767	L+D+F	1.363	1767	1864	1.00	0.95
LWS-1.2-45-150-60-12	6879	L+D+F	1.401	6022	6325	1.14	1.09
LWS-0.6-43-94-45-12	1278	L+D+F	1.531	1285	1331	1.00	0.96
LWS-0.6-90-150-60-12	2252	L+D+F	2.025	2202	2193	1.02	1.03
					Mean (P_m)	1.11	1.02
					COV (V_P)	0.063	0.054
					ϕ_b =0.8, Reliability index (β_1)	3.39	3.10
					ϕ_b =0.8, Reliability index (β_2)	3.19	2.90
					ϕ_b =0.9, Reliability index (β_1)	2.92	2.63
					ϕ_b =0.9, Reliability index (β_2)	2.72	2.43

L=Local buckling; D=Distortional buckling; F=Flexural buckling

Table 2: Comparison of moment capacities obtained from test and FEA results with DSM predictions for sections subjected to distortional buckling

Comparison of Moment Capacities obtained from Test and FEA Results with DSM Predictions

The moment capacities of the cold-formed steel stiffened channel sections subjected to four-point bending obtained from experimental investigation (M_{EXP}) and finite element analysis (M_{FEA}) were compared with the nominal moment capacities determined using the DSM (M_{DSM}) in NAS (2012) for cold-formed steel structures, as shown in Tables 1 and 2. The moment capacities of the three-point bending tests are higher than those of the four-point bending tests (Wang & Young, 2014). Therefore, the three-point bending tests were not used to ensure a conservative comparison. The mean value of experimental-to-predicted moment ratio (M_{EXP} /M_{DSM}) and FEA-to-predicted moment ratio (M_{FEA} /M_{DSM}) is 1.34 with the corresponding coefficient of variation (COV) of 0.165, and the calculated reliability index (β_1) and reliability index (β_2) are 3.53 and 3.36, respectively, for sections subjected to local buckling as shown in Table 1. For sections subjected to distortional buckling, the mean value of M_{EXP} /M_{DSM} and

M_{FEA} /M_{DSM} is 1.11 with the corresponding COV of 0.063, and the calculated β_1 and β_2 are 3.39 and 3.19, respectively, as shown in Table 2. The comparison of test and FEA results with predicted strengths by DSM is also plotted in Fig. 3(a) and Fig. 3(b) for local buckling and distortional buckling, respectively.

Modified Design Formulae for Local Buckling and Distortional Buckling

It is shown that the nominal moment capacities (M_{DSM}) predicted using the current DSM in NAS (2012) are quite conservative for the cold-formed steel beams with stiffened channel sections investigated in this study, especially for those specimens failed by local buckling. Therefore, the current direct strength formulae (Eq. (2) ~ Eq. (5)) are modified. The modified formulae for calculating the nominal flexural strength (M_{nl}) subjected to local buckling for sections symmetric about the axis of bending are as follows:

For $\lambda_l \le 0.880$, $$M_{nl} = [1 + (\eta - 1)(1 - 1/C_{yl}^2)]\ M_y \tag{6}$$

For $\lambda_l > 0.880$, $$M_{nl} = \left[1 - 0.06\left(\frac{M_{crl}}{M_y}\right)^{0.26}\right]\left(\frac{M_{crl}}{M_y}\right)^{0.26} M_y \tag{7}$$

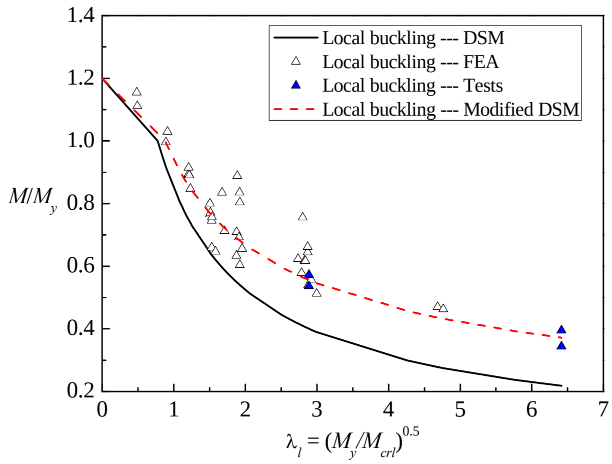
where $C_{yl} = \sqrt{0.880/\lambda_l} \le 3$, η is the shape factor depends on the shape of the cross-section ($\eta = Z_f / S_f$). It should be noted that the average value of η for the stiffened channel sections symmetric about the axis of bending investigated in this study is 1.2.

The modified formulae for calculating the nominal flexural strength (M_{nd}) subjected to distortional buckling for sections symmetric about the axis of bending are as follows:

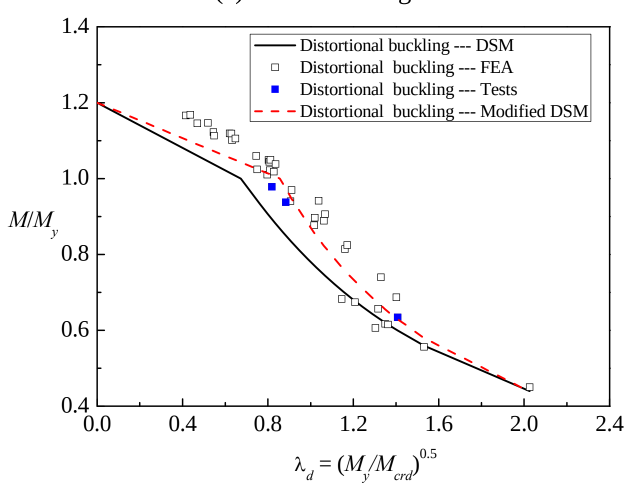
For $\lambda_d \le 0.857$, $$M_{nd} = [1 + (\eta - 1)(1 - 1/C_{yd}^2)]\ M_y \tag{8}$$

For $\lambda_d > 0.857$, $$M_{nd} = \left[1 - 0.13\left(\frac{M_{crd}}{M_y}\right)^{0.54}\right]\left(\frac{M_{crd}}{M_y}\right)^{0.54} M_y \tag{9}$$

where $C_{yd} = \sqrt{0.857/\lambda_d} \le 3$; the rest of the symbols in Eq. (6) - Eq. (9) are defined in Eq. (2) - Eq. (5). It should be noted that Eq. (6) and Eq. (8) are identical to Eq. (2) and Eq. (4), respectively, except the terms C_{yl} and C_{yd} are slightly different. The coefficient of 0.15 and exponent of 0.4 in Eq. (3) have been changed to 0.06 and 0.26 in Eq. (7), respectively. Subsequently, the value of slenderness λ_l has been modified from 0.776 to 0.880. Furthermore, the coefficient of 0.22 and exponent of 0.5 in Eq. (5) have been changed to 0.13 and 0.54 in Eq. (9), respectively, and the value of slenderness λ_d has also been modified from 0.673 to 0.857.



(a) Local buckling



(b) Distortional buckling

Figure 3: Comparison of DSM predicted strengths with test and FEA results

The nominal moment capacities (M_{DSM*}) of the cold-formed steel stiffened channel sections were calculated using the modified direct strength formulae (Eq. (1) and Eq. (6) - Eq. (9)). The comparison of the experimental and numerical data with the nominal values predicted by the modified DSM is shown in Tables 1 and 2. The mean value of experimental-to-predicted moment ratio (M_{EXP} /M_{DSM*}) and FEA-to-predicted moment ratio (M_{FEA} /M_{DSM*}) is 1.04 with the corresponding COV of 0.099, and the reliability index (β_1) and reliability index (β_2) are 3.01 and 2.82, respectively, for sections subjected to local buckling. For sections subjected to distortional buckling, the mean value of M_{EXP} /M_{DSM*} and M_{FEA} /M_{DSM*} is 1.02 with the corresponding COV of 0.054, and the values of β_1 and β_2 are 3.10 and 2.90, respectively. Furthermore, the reliability indices were also calculated for the modified DSM when the resistance factor (ϕ_b) of 0.9 was used. The reliability indices are β_1 = 2.55 and β_2 = 2.36 for sections subjected to local buckling as well as β_1 = 2.63 and β_2 = 2.43 for sections subjected to distortional buckling. The ratios of moment capacities over the yield moment (M/M_y) were plotted against the slenderness for sections failed by local buckling ($\lambda_l = \sqrt{M_y / M_{crl}}$) and sections failed by distortional buckling ($\lambda_d = \sqrt{M_y / M_{crd}}$), as shown in Fig. 3(a) and Fig. 3(b) respectively, where the moment capacities (M) were obtained from the experimental investigation (M_{EXP}), finite element analysis (M_{FEA}), DSM (M_{DSM}) and modified DSM (M_{DSM*}).

It is shown that the modified formulae of DSM are accurate and reliable with the reliability indices larger than the target reliability index ($\beta_0 = 2.5$) for sections failed by local buckling and sections failed by distortional buckling when the resistance factor (ϕ_b) of 0.8 is used. It should be noted that the reliability index (β_1) for load combination of 1.2 DL + 1.6 LL in the modified DSM is also larger than the target reliability index when the resistance factor (ϕ_b) of 0.9 was used, as shown in Tables 1 and 2. Hence, the modified direct strength formulae (Eq. (6) - Eq. (9)) are recommended for the design of cold-formed steel stiffened channel sections, and these sections can be potentially included in the prequalified sections subjected to bending in NAS (2012). The limitations of the beam sections for the modified DSM are summarized in Table 3.

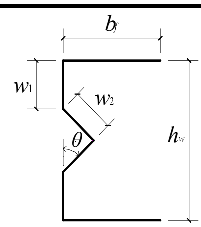
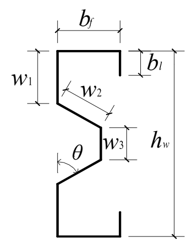
Sections	Geometric limitation
	$33 \le h_w / t \le 200$ $8.3 \le b_f / t \le 86.7$ $1.6 \le h_w / b_f \le 4$ $30^o \le \theta \le 45^o$ $f_y \le 600\text{MPa}$ $9 \le w_1 / t \le 200$
	$26 \le h_w / t \le 250$ $8.3 \le b_f / t \le 75$ $1.7 \le b_l / t \le 25$ $2.1 \le h_w / b_f \le 5$ $0.2 \le b_l / b_f \le 0.6$ $30^o \le \theta \le 60^o$ $f_y \le 590\text{MPa}$ $7 \le w_1 / t \le 90$

Table 3: Limitations of beam sections for modified DSM

Conclusions

This paper presents a parametric study of 75 beams with stiffened channel sections subjected to four-point bending. A non-linear finite element model was used in the parametric study that has been verified against experimental results. A total of 26 sections was investigated. Each section has two to three thicknesses that ranged from 0.48 to 3.6 mm in order to cover a wide range of section slenderness. The flange slenderness (b_f/t), overall web depth-to-thickness ratio (h_w/t), and the geometry of stiffeners in the channel sections were investigated. The moment capacities of the cold-formed steel stiffened channel sections subjected to four-point bending obtained from the tests and finite element analysis were compared with the nominal moment capacities determined using the current DSM in the North American Specification (NAS, 2012) for cold-formed steel structures. It is shown that the nominal moment capacities predicted using the current DSM are quite conservative for the cold-formed steel channels with stiffened web subjected to bending, especially for those specimens failed by local buckling. Therefore, the current direct strength formulae are modified in this study. It is shown that the modified DSM provides better predictions compared to the current DSM. Furthermore, the reliability analysis demonstrated that the modified formulae of DSM is reliable when the resistance factor (ϕ_b) of 0.9 was used. Thus, it is recommended to use the modified DSM for the design of cold-formed steel channels with stiffened web subjected to bending.

Appendix. – References

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Appendix. – Notation

b_f = width of flange
b_l = depth of lip
C_p = correction factor in reliability analysis

f_y = yield stress
F_m = mean value of fabrication factor
h_w = overall depth of web
M_{crd} = critical elastic distortional buckling moment
M_{crl} = critical elastic local buckling moment
M_{DSM} = nominal flexural strength predicted by current DSM
M_{DSM*} = nominal flexural strength predicted by modified DSM
M_{EXP} = moment capacities obtained from experimental investigation
M_{FEA} = moment capacities obtained from finite element analysis
M_m = mean value of material factor
M_{nd} = nominal flexural strength for distortional buckling
M_{ne} = nominal flexural strength for lateral-torsional buckling
M_{nl} = nominal flexural strength for local buckling
M_p = member plastic moment
M_y = member yield moment
P_m = mean value of experimental / FEA-to-predicted moment ratio
r_i = inner radius of the round corner of sections
S_f = gross section modulus referenced to the extreme fiber at first yield
t = thickness of steel plate with coating
V_F = coefficient of variation of fabrication factor
V_M = coefficient of variation of material factor
V_P = coefficient of variation of experimental / FEA-to-predicted moment ratio
w_1, w_2, w_3 = width of plate elements of stiffened channel sections
Z_f = plastic section modulus
β_0 = target reliability index
β_1 = reliability index using combination of 1.2 dead load + 1.6 live load
β_2 = reliability index using combination of 1.25 dead load + 1.5 live load
η = shape factor
θ = angle of inclined web element from the vertical axis
λ_d = slenderness for distortional buckling
λ_l = slenderness for local buckling
$\sigma_{0.2}$ = 0.2% proof stress (yield stress)
σ_{crd} = elastic distortional buckling stress
σ_{crl} = elastic local buckling stress
ϕ_b = resistance factor for beams