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Direct Strength Design of Cold-Formed C-Sections for Shear

Cao Hung Pham[1] and Gregory J. Hancock[2]

Abstract

The Direct Strength Method (DSM) recently included in the North American Specification and Australian/New Zealand Standard AS/NZS 4600:2005 gives design rules for compression and bending. No rules are presented at this stage for shear. Two series of tests on C-section can be used to develop and calibrate rules for design in shear. These are the University of Missouri Rolla tests of the 1970's and recent tests on high strength C-sections at the University of Sydney. Both series of tests use a similar test rig although different levels of tension field action have been observed. Two features researched are the effect of full section shear buckling (as opposed to web only shear buckling), and tension field action. Full section buckling is a feature of the DSM but requires software that can evaluate full sections for shear. The paper proposes DSM design rules for C-sections in shear both with and without tension field action. Both series of test results are compared with the proposed design rules.

1. INTRODUCTION

In both the Australian Standard and American Specification for the Design of Cold-Formed Steel Structures, which include the newly developed Direct Strength Method (DSM) of design, the method presented [Chapter 7 of AS/NZS 4600:2005 (Standards Australia, 2005)], Appendix 1 of the North American Specification (AISI, 2007)] is limited to pure compression and pure bending.

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The Direct Strength Method (Schafer and Peköz, 1998) was formally adopted in the North American Design Specification in 2004 and in Australian/New Zealand Standard AS/NZS 4600:2005 as an alternative to the traditional Effective Width Method (EWM) in 2005. It uses elastic buckling solutions for the entire member cross section to give the direct strength rather than for elements in isolation. The first advantage of the DSM is that it allows direct computation of the capacity of cold-formed thin-walled members of complex section shape (eg. with intermediate stiffeners). Secondly, the interaction between local and overall modes, distortional and overall modes is easily taken into account. The DSM uses numerical solutions for elastic buckling and requires computer software such as THIN-WALL (CASE, 2006) or CUFSM (Schafer and Ádány, 2006) to evaluate elastic buckling stresses. There is no need to calculate cumbersome effective sections especially with intermediate stiffeners.

The development of the DSM for columns and beams including the reliability of the method is well researched. In the review of the DSM of cold-formed steel member design, Schafer (2006) noted that no formal provisions for shear currently exist for the DSM. However, as recommended in the AISI Direct Strength Design Guide (AISI, 2006), the existing provisions in the North American Design Specification and AS/NZS 4600:2005 could be suitably modified into the DSM format.

In order to extend DSM to purlin systems for shear, and combined bending and shear, vacuum rig tests on continuous lapped cold-formed purlins at the University of Sydney over a 10 year period, have been used to calibrate DSM design proposals for shear and combined bending and shear (Pham and Hancock, 2009a). The conclusions from this calibration are that the existing bending and shear equations in AS/NZS 4600:2005 in DSM format will provide reliable designs irrespective of whether the limiting design moment in the interaction equation is based on the lesser of the local buckling and distortional buckling moments (called Proposal 1) or the local buckling moment alone (called Proposal 2).

To further investigate these proposals, additional tests on C-sections including tests on both plain C- and SupaCee® purlin sections (Lysaght, 2003) in predominantly shear, combined bending and shear, and bending only (Pham and Hancock, 2009b, 2009c) have been performed at the University of Sydney. The high strength SupaCee® profile steel sections contain additional return lips and web stiffeners which enhance the bending and shear capacity of the sections. They are used widely in Australia as purlins in roof and wall systems. The basic design of the test rig was developed at the University of Missouri-Rolla. In the 1970's, LaBoube and Yu (1978) conducted a series of tests including a total of forty three beam specimens subjected primarily to shear stress. They found that, for shear, the exact critical buckling load for beam webs is difficult to determine experimentally and the post-buckling strength of web elements due to tension field action increases as the h/t ratio of the web, the aspect ratio of the web, and the yielding point of the material increase. Further, the arrangement of connections has a significant effect on the ultimate shear capacity of the unreinforced webs. The current Effective Width Method for shear was calibrated against these tests.

The main objectives of this paper are:

- To summarise analyses of full sections in shear with a view to providing elastic shear buckling loads V_{cr} which can be used as input to the Direct Strength Method of design of complete sections in shear.
- To summarise both University of Sydney (UoS) and University of Missouri-Rolla (UMR) test results on cold-formed C- sections subjected primarily to shear.
- To summarise proposals for extension of the Direct Strength Method to shear. The proposals are made both with and without Tension Field Action (TFA) and are compared with both the UoS and UMR test results on the lipped C-sections.

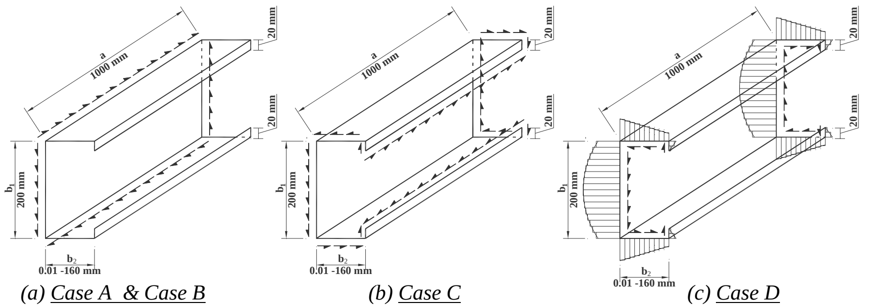
2. SHEAR BUCKLING OF FULL SECTIONS

Analysis of the shear buckling stress of flat rectangular plates has been performed by many investigators (Timoshenko and Gere, 1961; Bulson, 1970; Bleich, 1952). The traditional approach has been to investigate shear plate buckling in the web alone and to ignore the behaviour of the whole section including the flanges. There does not appear to be any consistent theoretical or experimental investigation of the whole section buckling of thin-walled sections under shear. Recently, Pham and Hancock provided solutions to the shear buckling of complete channel sections (2009d) and plain C- lipped sections with an intermediate web stiffener (2009e) loaded in pure shear parallel with the web by using a Spline Finite Strip Method.

The Spline Finite Strip Method (SFSM) is a development of the semi-analytical finite strip method originally derived by Cheung (1976). It uses spline functions in the longitudinal direction in place of the single half sine wave over the length of the section, and has been proven to be an efficient tool for analyzing structures with constant geometric properties in a particular direction, generally the longitudinal one. The advantage of the spline finite strip analysis is that it allows more complex types of loading and boundary conditions other than simple supports to be easily investigated and buckling in shear is also easily accounted for. Initially, the spline finite strip method was fully developed for the linear elastic structural analysis of folded plate structures by Fan and Cheung (1982).

The SFSM was then extended to buckling and nonlinear analyses of flat plates and folded-plate structures by Lau and Hancock (1986) and Kwon and Hancock (1991, 1993). The spline finite strip method involves subdividing a thin-walled member into longitudinal strips where each strip is assumed to be free to deform both in its plane (membrane displacements) and out of its plane (flexural displacements). The functions used in the longitudinal direction are normally B3 splines. The ends of the section under study are normally free to deform longitudinally but are prevented from deforming in a cross-sectional plane.

For the shear buckling analyses described by Pham and Hancock (2009d), three different methods, which represent different ways of incorporating the shear stresses in the thin-walled section, are used in this analysis as shown in Fig. 1.



(a) Case A & Case B *(b) Case C* *(c) Case D*

Figure 1. Shear Flow Distributions in Lipped Channels

These include pure shear in the web only (Cases A and B), pure shear in the web and the flanges (Case C) , and a shear distribution similar to that which occurs in practice allowing for section shear flow (Case D). The stress states studied are not in equilibrium as shear can only be generated in a section by moment gradient. However, the studies allow the shear buckling to be isolated and investigated. Fig 2 from Pham and Hancock (2009d) shows the results of the buckling analyses of the lipped channel section of length a=1000 mm and the ratios of flange to web width (b_2/b_1) from 0.00005 to 0.8 (Case D). The corresponding buckling mode shapes are shown in Fig 3 (Case D). They demonstrate a range of buckling modes including section twisting for sections with narrow flanges, flange distortional buckling and web shear buckling depending upon the width of the flanges.

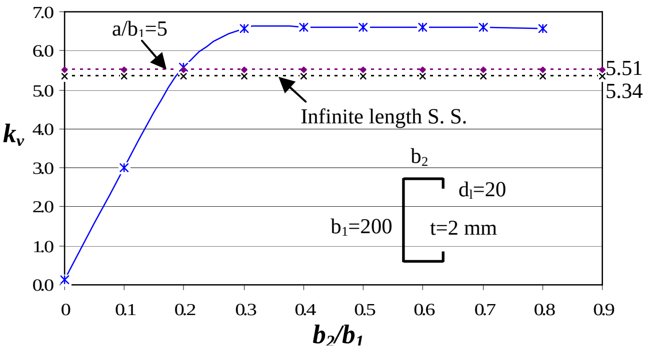


Figure 2. The Ratio of Flange and Web Widths *(b_2/b_1)* and The Shear Buckling Coefficients *(k_v)* for Case D

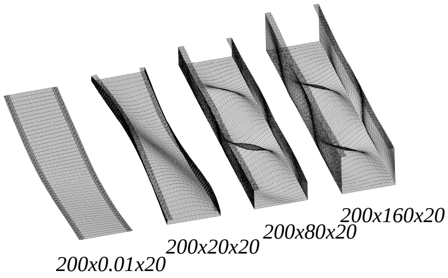


Figure 3. Buckling Mode Shape of Lipped Channel Section Length = 1000 mm, $a/b_1=5$ for Case D

3. EXPERIMENTS ON PLAIN C- LIPPED SECTIONS AND SUPACEE® SECTIONS IN SHEAR

3.1 Experimental Rig and Tests Specimens

The experimental program comprised a total of thirty six tests conducted in the J. W. Roderick Laboratory for Materials and Structures at the University of Sydney. All tests were performed in the 2000 kN capacity DARTEC testing machine, using a servo-controlled hydraulic ram. Two different commercially available plain C- lipped and SupaCee® sections of 150 and 200 mm depths as shown in Fig. 4 were chosen with three different thicknesses of 1.5, 1.9 and 2.4 mm (for plain C- lipped sections) and 1.2, 1.5 and 2.4 mm (for plain C- lipped sections).

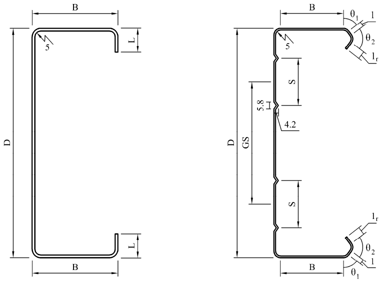


Figure 4. Dimensions of Plain and SupaCee® Channel Sections

The average measured dimensions and properties for the *V* series of both plain C-lipped and SupaCee® sections are given in Pham and Hancock (2009b, 2009c) respectively. A diagram of the test set-up is shown in Fig. 5 for shear *(V)* series. Although the tests described in LaBoube and Yu (1978) contained straps at the loading points as described later, tests both with and without straps are included in the test program described in this paper.

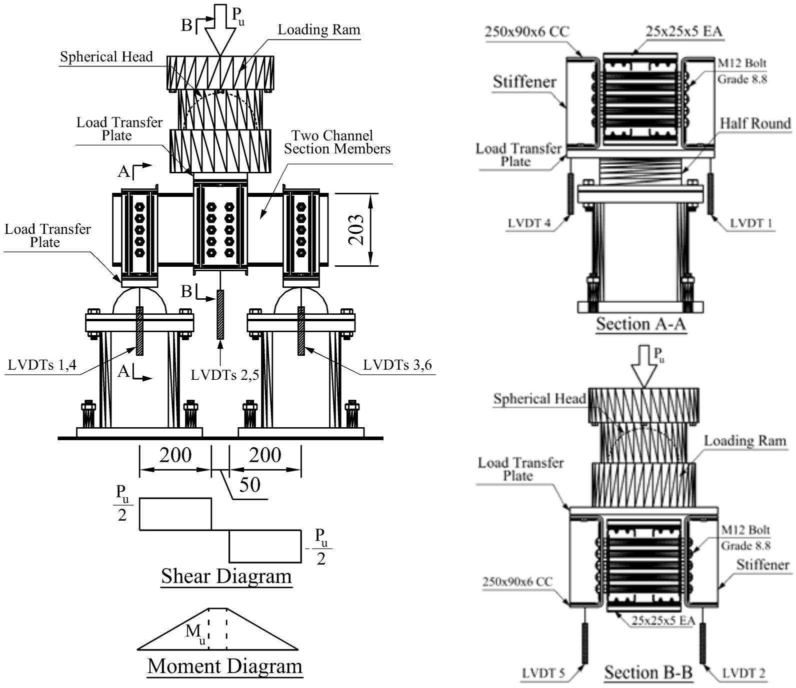
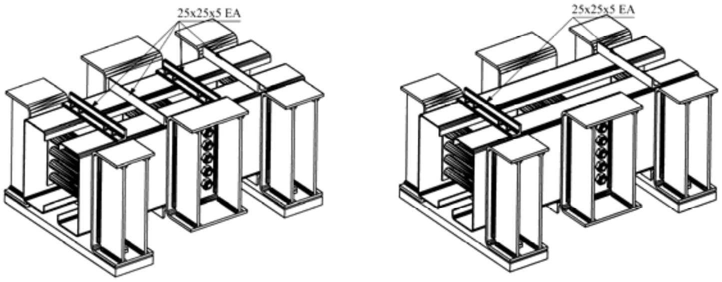


Figure 5. *V* Test Series Configuration (Dimensions for 200 mm Deep Section)

3.2 Test With and Without Straps Configurations

Twenty four tests (18 of plain lipped C- sections and 6 of SupaCee® sections) of the thirty six tests had four 25x25x5EA straps connected by self-tapping screws on each of the top and bottom flanges adjacent to the loading point and reactions as shown in Fig. 6(a). Twelve remaining tests (6 of plain lipped C- sections and 6 of SupaCee® sections) were tested without the two 25x25x5EA straps adjacent to the loading points on the top flange as shown in Fig. 6(b). The purpose of these two straps is to prevent distortion of the top flanges under compression caused by bending moment. The distortion may be a consequence of unbalanced shear flow or distortional buckling.



(a) With Straps (b) Without Straps

Figure 6. *V* Test Series Configuration With and Without Straps

3.3 University of Sydney (UoS) Plain Lipped C- and SupaCee® Section Shear Test Results

The full set of test results for the plain lipped C- and SupaCee® sections is given in the research reports by Pham and Hancock (2009b, 2009c). The tests results are compared with existing design methods in AS/NZS 4600:2005 and with the tension field action included using the rules of AS 4100:1998 (Standards Australia 1998). All slender section specimens in the *V* series were found to develop tension field action as the connection bolts at the loading and support points extended over the full depth of the section whether 150 mm (4 bolts) or 200 mm (5 bolts). The inclusion of straps attached to the flanges to prevent distortion at the loading and support points further enhanced the tension field action. They are compared with DSM design proposals in Section 3 following. All results of UoS tests for the predominantly shear *(V)* test series of both the plain C- sections and SupaCee® sections are summarized in Table 1.

Test	Section	V_T (kN)	V_y(AISI) (kN)	V_{cr} (kN)	$\sqrt{V_y / V_{cr}}$	V_T / V_y
V1	C15015	55.43	68.37	43.13	1.259	0.811
V2	C15015	56.08	68.26	43.21	1.257	0.822
V3	C15015	54.47	68.36	43.14	1.259	0.797
Vw	C15015	51.28	68.29	43.19	1.257	0.751
V1	C15019	76.78	84.99	88.23	0.982	0.903
V2	C15019	75.65	85.07	88.15	0.982	0.889
V3	C15019	77.85	84.95	88.27	0.981	0.916
Vw	C15019	70.87	84.85	88.37	0.980	0.835
V1	C15024	94.24	96.86	178.96	0.736	0.973
V2	C15024	96.04	96.73	179.19	0.735	0.993
V3	C15024	95.56	96.92	178.84	0.736	0.986
Vw	C15024	93.38	96.62	179.40	0.734	0.966
V1	C20015	56.14	88.58	31.84	1.668	0.634
V2	C20015	53.89	88.51	31.87	1.667	0.609
V3	C20015	57.76	88.56	31.85	1.667	0.652
Vw	C20015	50.82	88.61	31.83	1.669	0.573
V1	C20019	86.51	109.65	65.84	1.290	0.789
V2	C20019	86.06	109.66	65.83	1.291	0.785
V3	C20019	83.38	109.48	65.94	1.289	0.762
Vw	C20019	75.82	109.50	65.93	1.289	0.692
V1	C20024	115.45	131.69	132.19	0.998	0.877
V2	C20024	113.60	131.27	132.61	0.995	0.865
V3	C20024	112.61	130.92	132.96	0.992	0.860
Vw	C20024	103.31	131.81	132.07	0.999	0.784
V1	SC15012	42.13	59.93	27.00	1.490	0.703
Vw	SC15012	39.33	60.08	26.94	1.493	0.655
V1	SC15015	55.58	67.10	53.32	1.122	0.828
Vw	SC15015	51.87	67.58	52.94	1.130	0.768
V1	SC15024	97.99	102.71	219.62	0.684	0.954
Vw	SC15024	92.92	102.69	219.66	0.684	0.905
V1	SC20012	46.48	82.47	18.95	2.086	0.564
Vw	SC20012	45.55	82.57	18.92	2.089	0.552
V1	SC20015	62.07	91.35	37.44	1.562	0.679
Vw	SC20015	61.65	91.40	37.42	1.563	0.675
V1	SC20024	124.21	137.70	154.52	0.944	0.902
Vw	SC20024	117.31	137.04	155.26	0.939	0.856

Table 1. (UoS) *V* Series Test Results of Plain C- and SupaCee$^{®}$ Sections

3.4 University of Missouri-Rolla (UMR) Plain Lipped C- Section Shear Test Results

The full set of test results for the plain lipped C- sections is given in the research report by LaBoube and Yu (1978). A total of forty three shear tests have been conducted to determine the structural behavior of web elements subjected primarily to shear. All results of tests for the UMR shear *(V)* test series are summarized in Table 2.

In the report of LaBoube and Yu (1978), specimen Nos. S-10-1, S-10-2 and S-10-3 have been discarded due to combined bending and shear failure. Specimen No. S-9-3 has been discarded due to premature failure, and specimen Nos. S-19-1, S-19-2, S-20-1 and S-20-2 have been discarded due to web crippling, which resulted at the end supports because the connection rods were poorly arranged. They are therefore not included in Table 2. As can also be seen in Table 2, six additional tests (Nos. MS-2-1, MS-2-2, MS-3-1, MS-3-2, MS-8-1, MS-8-2) were performed with cover plates added to both the tension and compression flanges. In Table 2, the k_v values for the MS sections are simply those of the plain sections, as the restraint from the intermittently connected cover plates is difficult to quantify.

Beam Specimen	V_T **(kN)**	V_y**(AISI) (kN)**	k_v **Section**	V_{cr} **(kN)**	$\sqrt{V_y / V_{cr}}$	V_T / V_y
S-1-1	19.10	19.08	25.741	62.88	0.551	1.001
S-1-2	19.16	18.94	22.893	57.30	0.575	1.011
S-2-1	17.21	23.27	9.742	20.20	1.073	0.740
S-2-2	17.38	23.44	9.834	20.96	1.057	0.742
S-3-1	17.79	27.99	9.811	16.62	1.298	0.636
S-3-2	18.78	27.53	9.777	16.55	1.290	0.682
S-8-1	16.67	23.34	9.944	20.21	1.075	0.714
S-8-2	16.85	23.46	9.874	19.96	1.084	0.718
S-9-1	16.88	27.53	9.781	16.84	1.279	0.613
S-9-2	18.68	27.76	9.824	16.78	1.286	0.673
S-9-4	17.08	27.44	9.818	17.26	1.261	0.623
S-9-5	15.88	27.70	9.791	17.20	1.269	0.573
S-9-6	18.35	27.97	9.920	17.56	1.262	0.656
S-9-7	12.92	27.92	6.790	12.15	1.516	0.463
S-9-8	13.57	27.87	6.790	12.06	1.520	0.487
S-10-4	25.35	27.33	17.458	60.16	0.674	0.928
S-10-5	24.89	27.40	17.587	60.45	0.673	0.908
S-11-1	25.58	41.01	9.752	25.30	1.273	0.624
S-11-2	28.83	40.63	9.741	25.51	1.262	0.710
S-11-3	27.33	41.26	9.779	26.24	1.254	0.662
S-12-1	20.02	51.17	9.558	20.19	1.592	0.391
S-12-2	23.89	51.47	9.585	20.13	1.599	0.464
S-12-3	20.59	53.00	9.603	21.70	1.563	0.389
S-17-1	27.36	41.20	9.670	27.03	1.235	0.664
S-17-2	26.69	41.20	9.702	27.12	1.233	0.648
S-18-1	26.79	52.50	9.919	22.63	1.523	0.510
S-18-2	24.35	52.64	9.842	22.39	1.533	0.463
S-19-3	18.40	55.55	9.552	13.53	2.027	0.331
S-20-3	15.45	68.72	9.438	10.20	2.596	0.225
MS-2-1	17.50	23.35	9.758	19.81	1.086	0.750
MS-2-2	17.76	22.92	9.810	19.43	1.086	0.775
MS-3-1	17.51	27.92	9.815	16.67	1.294	0.627
MS-3-2	16.48	27.90	9.844	16.73	1.292	0.591
MS-8-1	18.03	23.18	9.943	20.34	1.068	0.778
MS-8-2	16.24	23.52	9.936	20.91	1.061	0.690

Table 2. (UMR) *V* Series Test Results of Plain C- Sections

4. PROPOSED DIRECT STRENGTH METHOD (DSM) OF DESIGN OF COLD- FORMED C- SECTIONS FOR SHEAR

4.1 Nominal Shear Capacity Based on AISI in DSM Format in Shear without Tension Field Action

The equations in Section C3.2.1 of the North American Specification (AISI, 2007) which are expressed in terms of a nominal shear stress F_v have been changed to DSM format by replacing stresses by loads as follows:

$$\text{For } \lambda_v \leq 0.815 \; : \; V_v = V_y \tag{1}$$

$$\text{For } \; : \; 0.815 < \lambda_v \leq 1.227 \; : \; V_v = 0.815\sqrt{V_{cr}V_y} \tag{2}$$

$$\text{For } \; : \; \lambda_v > 1.227 \; V_v = V_{cr} \tag{3}$$

where $\lambda_v = \sqrt{V_y / V_{cr}}$, V_y = yield load of web = $0.6 A_w f_y$,

V_{cr} = elastic shear buckling force of web = $\dfrac{k_v \pi^2 E A_w}{12\left(1-\nu^2\right)\left(\dfrac{d_1}{t_w}\right)^2}$

d_1 = depth of the flat portion of the web measured along web plane,

t_w = thickness of web, A_w = area of web = $d_1 \times t_w$,

k_v = shear buckling coefficient for whole sections (as shown in Section 2)

To account for the shear buckling of the whole section rather than simply the web, the shear buckling coefficient *(k_v)* can be back-calculated from the shear buckling load V_{cr} (as summarized in Tables 1, 2 for both UoS and UMR test results) of the whole section as described in Pham and Hancock (2009d) (also in Section 2) by using the Spline Finite Strip Method. In this way, the DSM philosophy of section rather than element buckling can now be incorporated in the nominal shear capacity.

For the plain lipped C- sections, the computed values of the shear buckling coefficients *(k_v)* for the whole section increase from the theoretical value of a simply supported rectangular plate in shear of 9.34 for a Span:Panel Depth of 1:1 to 9.926 and 10.006 for the 150mm and 200 mm depth sections respectively. For the SupaCee® sections, the corresponding values are 12.204 and 11.709 as a result of the longitudinal intermediate stiffeners in the web.

4.2 Direct Strength Method based on AISI in DSM Format in Shear with Tension Field Action

The DSM nominal shear capacity *(V_v)* including Tension Field Action (TFA) is proposed based on the local buckling *(M_{sl})* equation where M_{sl}, M_{ol} and M_y are replaced by V_v, V_{cr} and V_y respectively as follows:

$$V_v = \left[1 - 0.15\left(\frac{V_{cr}}{V_y}\right)^{0.4}\right]\left(\frac{V_{cr}}{V_y}\right)^{0.4} V_y \tag{4}$$

where V_y is yield load of web $V_y = 0.6 A_w f_y$,

V_{cr} is elastic shear buckling force of web $V_{cr} = \dfrac{k_v \pi^2 E A_w}{12\left(1-\nu^2\right)\left(\dfrac{d_1}{t_w}\right)^2}$,

k_v = shear buckling coefficient for whole sections (as shown in Section 2)

The results of tests for the predominantly shear *(V)* test series of both UoS and UMR are plotted in Fig. 7. The TFA curve (Basler, 1961), the elastic buckling curve *(V_{cr})* and the DSM proposed curves for shear with and without TFA are also graphically reproduced in Fig. 7.

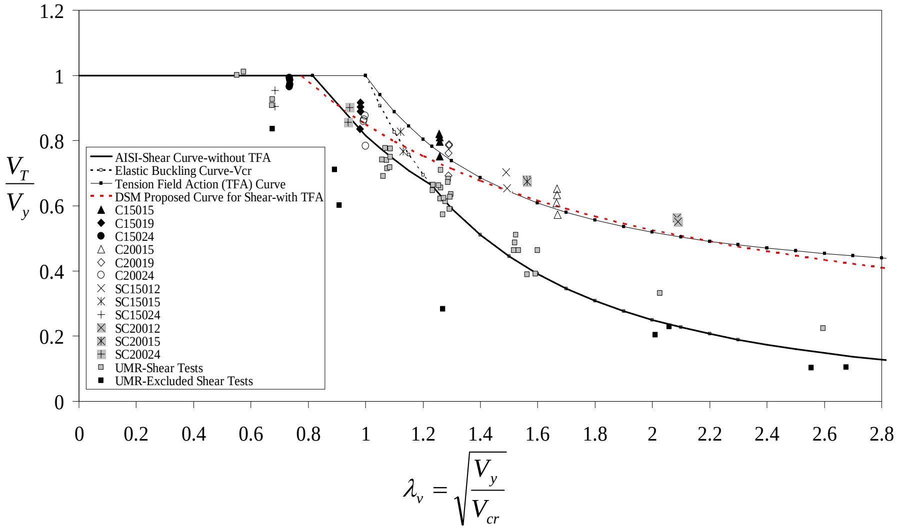
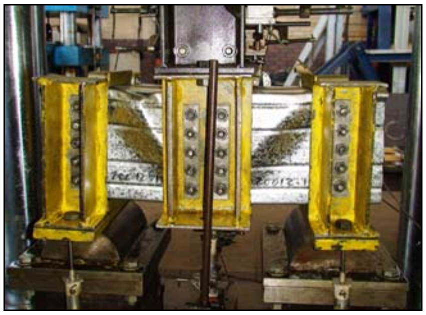


Figure 7. DSM Proposed Shear Curves and Shear Test Data (UoS and UMR)

Fig. 7 shows that all plain lipped C- and SupaCee® *V*-series tests of UoS lie close to the proposed DSM nominal shear capacity with TFA (see Eq. 4). The DSM proposed shear equation with TFA therefore gives a good mean fit to the *V*-series tests. They lie well above the AISI in DSM format equations (see Eqs. 1–3) presumably because significant tension field action was developed. Figs 8(a) and 8(b) show the corresponding buckling mode shapes of the SupaCee® section members with and without straps respectively for the *V*-series.



(a) With Straps

(b) Without Straps

Figure 8. Buckling Mode Shape of SupaCee® Section Members
V-Series – With and Without Straps

Fig. 7 also shows the UMR tests lie closer to the AISI in DSM format without TFA (see Eqs. 1-3). The reason for this is probably the inability of the deeper (and hence more slender) UMR sections to develop tension field action. This may be a result of keeping the same number of bolts for deeper sections although the test report does not fully specify the bolt configuration.

The mean (P_m) of the test results divided by the design model for each test series, and each design model (with and without TFA) is summarized in Table 3. The corresponding coefficient of variations (V_P) is also included in Table 3. The University of Sydney results are better predicted (P_m=1.022, V_P=0.068) by the DSM shear proposal with TFA, and the UMR results are better predicted (P_m=1.045, V_P=0.118) by the DSM shear proposal without TFA.

Test Program	**(Eqs. 1-3) DSM without TFA**		**(Eq. 4) DSM with TFA**	
	P_m	**V_P**	**P_m**	**V_P**
University of Sydney (Table 1)	1.293	0.295	1.022	0.068
University of Missouri-Rolla (Table 2)	1.045	0.118	0.835	0.128

Table 3. Comparison of P_m and V_P for UoS and UMR between
DSM Shear Proposals with and without TFA

The development of TFA in the University of Sydney may be a result of the bolts connecting the webs of the channels spanning the full depth of the section for both 150 mm and 200 mm tests. The two vertical rows of bolts have increased the restraints to the web panel and act as web stiffeners. These increased restraints have improved the post-buckling strengths of the web for the *V*-series. It is interesting to note that the slender sections (e.g. C20015, SC15012, SC20012 and SC20015) are more conservative than stockier sections. This fact shows that the more slender sections have more tension field action contribution to the ultimate strength of the sections in shear.

3. CONCLUSION

The paper has presented proposals for the design of cold-formed steel sections in shear by the Direct Strength Method (DSM) as in the North American Specification and Australian/ New Zealand Standard. The proposals are compared with tests in predominantly shear of both the University of Missouri Rolla C-section tests of the 1970's and recent tests on high strength plain lipped C- and SupaCee® sections at the University of Sydney. One feature of the DSM is that the full section buckling including intermediate stiffeners under shear has been included rather than simple web buckling in shear. This proposal produces better correlation with the test data but requires special computer software such as the Spline Finite Strip Method (SFSM) to determine the shear buckling loads of full sections. The shear capacity, V_v, is based on the DSM proposals with and without tension field action. The shear buckling load, V_{cr}, used in the DSM equations is based on the shear buckling coefficient of the full section and not just the web buckling in shear. The tests from the University of Sydney show that the DSM proposal curve for shear with tension field action gives a good mean fit to the shear tests and more accurate prediction on post-bucking strength of the C-sections in shear. The tests from the University of Missouri Rolla are better predicted by the DSM proposal curve for shear without tension field action. The increased restraints created by full bolt connections along the depth of the web panel at the supports and loading point may improve the post-buckling strengths of the web in shear. Research on this aspect is ongoing.

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