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Effect of Web Holes and Bearing Stiffeners on Flexural-Shear Interaction Strength of Steel Cold-Formed C-Channel Sections

Abstract

This paper presents an investigation on interaction equation between the required flexural strength, M, and the required shear strength, V, of cold-formed C-channels with web holes and bearing stiffeners. The primarily shear condition test was employed to study total 8 back to back lipped C channel sections of 95 and 100 mm depth when bearing stiffeners and circular holes were placed at center and both ends of specimens. The interaction equation were evaluated via Direct Strength Method, DSM, in accordance with the American Iron and Steel Institute for the design of cold-formed steel structural members, AISI 2007. A nonlinear finite element model was developed and verified against the test results in terms of failure buckling modes. It was concluded that the M-V interaction equation for specimens with web stiffeners was conservative where these specimens experienced plastic failure mode rather than local (Msl) or distortional (Msd) buckling mode. Moreover, the results indicated that proposed M-V interaction equation calculated by local buckling strength (Msl) adequately predicted the behavior of specimens with circular web holes.

Keywords

cold-formed steel; pure bending test; Direct Strength Method; inelastic buckling strength; local buckling; distortional buckling; non-dimensional slenderness.

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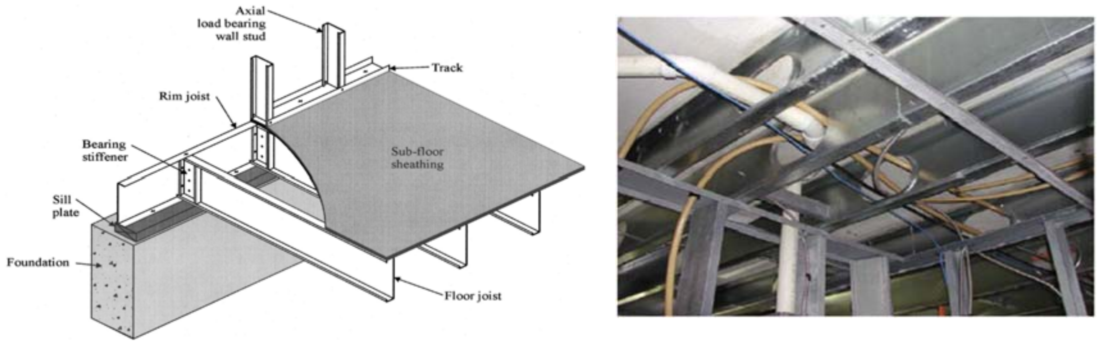
1 INTRODUCTION

Cold-formed steel joists are typically C-channel sections where they usually experience web buckling (or web crippling) under relatively low concentrated loads that usually occur at supports. To preclude such drawback resulted to web crippling, web bearing stiffeners are attached to the beam sections in order to transfer the vertical loads. Generally, smaller C-channel sections are considered as web bearing stiffeners which are attached to the back of the sections or install fit to fit between two flanges of C-sections as shown in Figure 1(a). The self-drilling screws are employ to connect the bearing stiffeners and web's sections. The stiffeners resist to the concentrated vertical loads at the preliminary stage of loading protocol, but web crippling occur when the section is expose to sufficiently high load. After this stage, the web bearing stiffeners act as short columns to transferring axial load from the top flange surface to the bottom flange.

Moreover, to facilitate electrical, plumbing and heating pipe the circular holes are placed in the webs of joists. Therefore, web crushing or crippling at this points are common failure mode of C-channel sections where they are subjected to significant concentrated load as this sections are not stiffened against this type of loading. Fig. 1(b) shows one example of floor joists with web that demand web crippling phenomenon to be taken into account on its design procedure.



(a) (b)

Figure 1: Bearing stiffeners (a) and web holes (b) at steel cold-formed joists.

The short span and cantilever beams are subjected to high bending and shear stress at their support regions. These beam webs shall resist against buckling as a result of combination of bending and shear stresses. For beams with unreinforced webs, the required flexural strength, M, and required shear strength, V, shall satisfy the following interaction equation in accordance with section C3.3 of American Iron and Steel Institute, AISI 2007:

$$\left\{\frac{M}{\emptyset_b M_n}\right\}^2 + \left\{\frac{V}{\emptyset_v V_n}\right\}^2 \leq 1.0 \tag{1}$$

For beams that have transverse web stiffeners, once $M/(\emptyset_b M_n) > 0.5$ and $V/(\emptyset_v V_n) > 0.7$, the following interaction equation in accordance with section C3.3 of AISI 2007 must be satisfied by M and V:

$$0.6\left\{\frac{M}{\emptyset_b M_n}\right\} + \left\{\frac{V}{\emptyset_v V_n}\right\} \leq 1.3 \tag{2}$$

where:

Mn= nominal flexural strength once bending alone is taken into account
M= required flexural strength

ϕ_b= resistance factor for bending (0.9)
V_n= nominal shear strength once shear alone is taken into account
V= required shear strength
Φ_v= resistance factor for shear (0.95)

The interaction equation for unreinforced web and web with transverse stiffeners is demonstrated in Figure 2. This specific interaction is resulted from an approximation of theoretical interaction of local buckling due to shear and bending that was calculated by Timoshenko and Gere (1961).

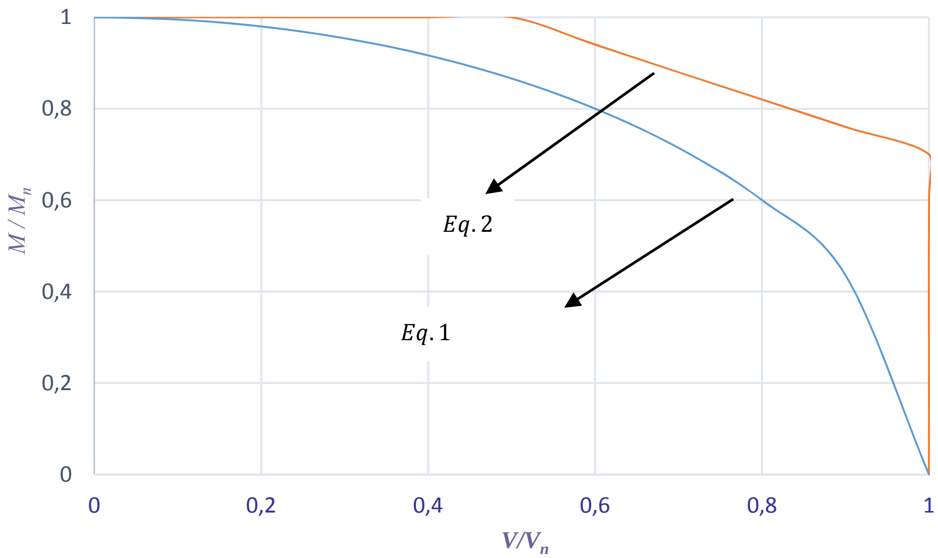


Figure 2: The interaction equation for unreinforced web and web with transverse stiffeners.

In order to investigate the structural performance of reinforced and unreinforced beam webs under shear stress, bending stress, web crippling load and combinations thereof, a research titled "Webs for Cold-Formed Steel Flexural Members" and funded by the American Iron and Steel Institute was conducted at the University of Missouri-Rolla. (Laboube and Yu 1978) and (Hetrakul and Yu 1978) are the two references that entail the results of this project. Based on their work, the load carrying capacity of transverse stiffeners was calculated according to column formulae that considered an adjacent portion of the web as a part of the stiffener column. Finding this effective portion of the web was a difficult task for analytical analysis because factors like elastic and inelastic instability of the stiffener, web crippling strength of a combination of stiffener and beam web and finally plate element local buckling of the stiffener have to be involved in this process. In later studies aimed at developing design rules for thin-walled members and/or stiffened plates, (Hoon, Rhodes et al. 1993) and (Bernard, Bridge et al. 1993) published their experimental investigations results. In another study, some modifications to the safety and design checking procedures of stiffened members were recommended by (Schafer 2002) to investigate the possibility of using intermediate stiffeners within the cross-section walls. The experimental and numerical work conducted by (Narayanan and Mahendran 2003), (Teter and Kolakowski 2004) and (Yang and Hancock 2004) regarding the load-carrying capacity and post-buckling behavior of cold-formed steel lipped channel columns having web and flange intermediate stiffeners is of great importance.

The shear capacity of beam sections with web openings could be reduced as a result of reduced web area. The significant shear capacity reduction of lipped channel beams will occur due to primary shear resisting area reductions. Yet, web openings do not affect the flexural capacity since they are commonly placed at web centers. Numerous variables influence the shear capacity of members with web openings like location, size and shape of web openings along with slenderness of web elements. Previous studies of researchers like (LaBoube, Yu et al. 1997), (Eiler, LaBoube et al. 1997) regarding the strength and shear behavior of cold-formed steel sections having web openings were only focused on to 'C' sections. (LaBoube, Yu et al. 1997) suggested a reduction factor (qs) to calculate the nominal shear capacity of cold-formed lipped channel beams having web openings. This factor was applied to the solid web strength of the shear element. LaBoube et al.'s work was continued and improved by (Eiler, LaBoube et al. 1997) with inclusion of the performance of web elements with openings under linearly varying shear force. In his tests, a uniform load with no constant shear was applied to cold-formed steel beams with web openings.

The shear performance of hollow flange channel beams aka Lite Steel beams having circular web openings were experimentally and numerically investigated by (Poologanathan and Mahendran 2012) (Keerthan and Mahendran 2012). In their studies, design equations regarding the shear capacity with web openings (Vnl) were developed according to a reduction factor qs applied to the shear capacity of Lite Steel beams without web openings (Vv). An isoparametric spline finite strip procedure was hired by (Pham and Hancock 2009) to study the elastic buckling of lipped and un-lipped channel sections under shear stresses. Their work included the influence of flanges over the shear buckling of web elements of channel sections. In a more recent study, 84 tests were conducted by (Zhou and Young 2010) on aluminium alloy square hollow sections having circular holes placed at the bottom center of the bearing plates. Two loading conditions namely Interior-Two-Flange (ITF) and End-Two-Flange (ETF) were used in web crippling tests. Also, they proposed a few reduction factors as well.

2 EXPERIMENTAL INVESTIGATION

2.1 Test Specimens

Lipped C-channel Sections with 150 and 100 mm depth were used for primarily shear condition tests. For each sections three different category were considered introduced as specimen with fit to fit stiffener, specimen with stiffener where there was a gap between flanges and stiffener, specimens with web circular hole and finally specimens without stiffener and hole. The circular holes and web bearing stiffeners installed inside the lipped C-channel sections at center and both supports. The dimensions of the two lipped C-channel sections are illustrated in Table 1 and Figure 3 where, L=overall depth of the lip; B= overall width of the flange; t= thickness of individual sections and D=total depth

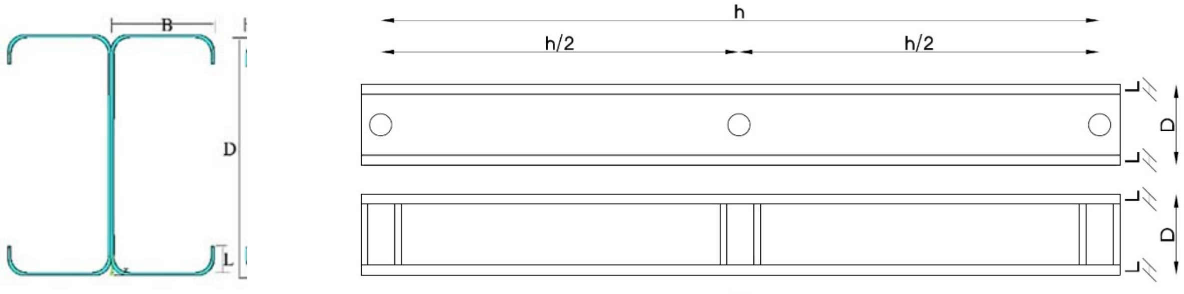


Figure 3: (a) Cross section of back to back lipped C-channel Section, (b) location of circular hole and web bearing stiffeners.

Section	Thickness (mm)	D (mm)	B (mm)	L (mm)
SC 150-16	1.6	150	130	16
SC 100-10	1.0	100	50	12

Table 1: Lipped C-channel sections' dimensions.

Two specimens were longitudinally considered from the web center of each C-channel to run the tensile tests. The tensile specimen's dimensions were in conformity with ASTM E 8 – E8M using a gauge with a width and length of 12.5 and 50 mm, respectively (Faridmehr, Osman et al. 2014). Table 2 illustrates the material properties of tested specimens based on tensile test results.

Specimen	E (GPa)	Fy at (0.2%) (MPa)	%Strain at 0.2% Yield	Fu (MPa)	%Strain at Maximum Stress
SC 150-16	192.02	478.01	0.448	510.59	6.82
SC 100-10	207.43	636.64	0.511	642.22	1.263

Table 2: Material properties of tested specimens.

2.2 Test Rig and Operation

The experimental phase of the study was conducted at the Structures and Materials Laboratory, Universiti Teknologi Malaysia, UTM. A reacting frame having a 500-kN capacity hydraulic jack and a 300-kN capacity load-cell was used to conduct the tests. The modeling setup in this study was considered for primarily shear condition test. The load was transferred to four 250×90×6 channel sections attached to the joists via two rows of M18 high tensile bolts in a vertical manner. Figures 4 highlight the test setup diagram along with an overview of primarily shear and pure bending conditions.

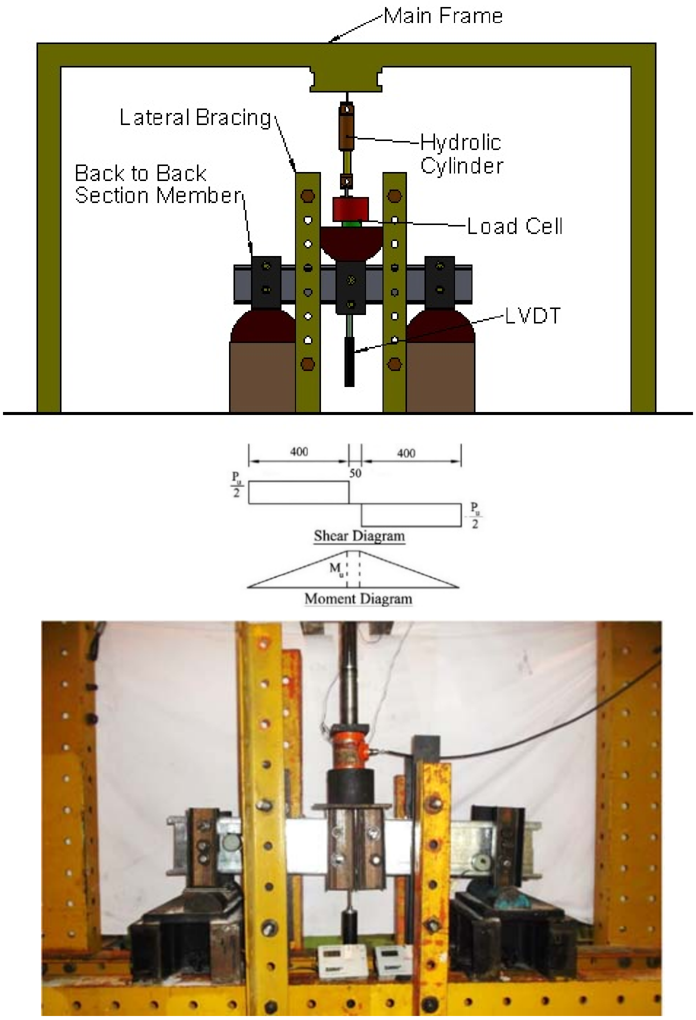


Figure 4: Primarily shear condition test setup configuration.

2.3 Test Results

The load against vertical displacement curves of primarily shear condition tests for SC150-16 and SC100-10 are graphically produced respectively in Figures 5 and 6. Based on Figures 5 and 6, the loads increased in a linear manner compared to vertical displacements for all specimens. However, for sections without bearing web stiffeners, sudden failure of specimens occurred within the elastic distortional buckling mode. In sections with fit to fit bearing web stiffeners, the load versus displacement curves were far away from the linear region and demonstrated a more nonlinear behavior once they reached to the peak loads. As it shown in both Figures 5 and 6 for specimens possess web bearing stiffeners with gap between joist flanges, the graph experienced a jump after reached to the pic points. In these specimens at the preliminary stage of loading there was not any contact between stiffeners and joist sections and the whole load tolerated by joist sections. At higher load the flanges immediately started to rotate and transfer the loads to the web bearing stiffeners. The flange rotation will also create moments at the top and bottom of the joist web that will start to curve the web outward as shown in Figure 7. Moreover, the significant nonlinear behavior for sections with web bearing stiffeners proved that local buckling was the governing failure mode in them.

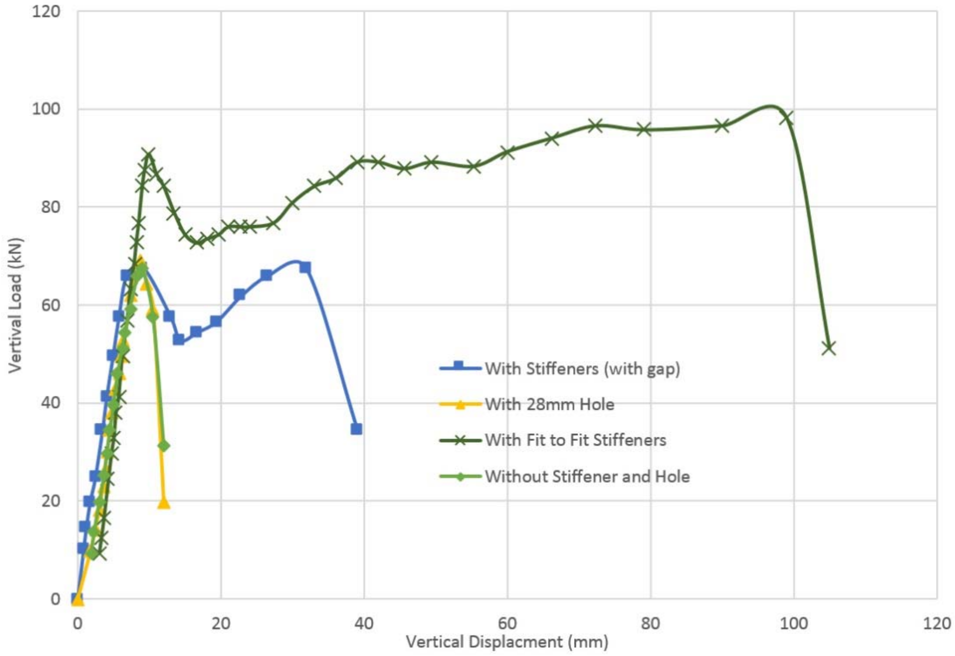


Figure 5: Load versus vertical displacement for SC 150-16 specimens.

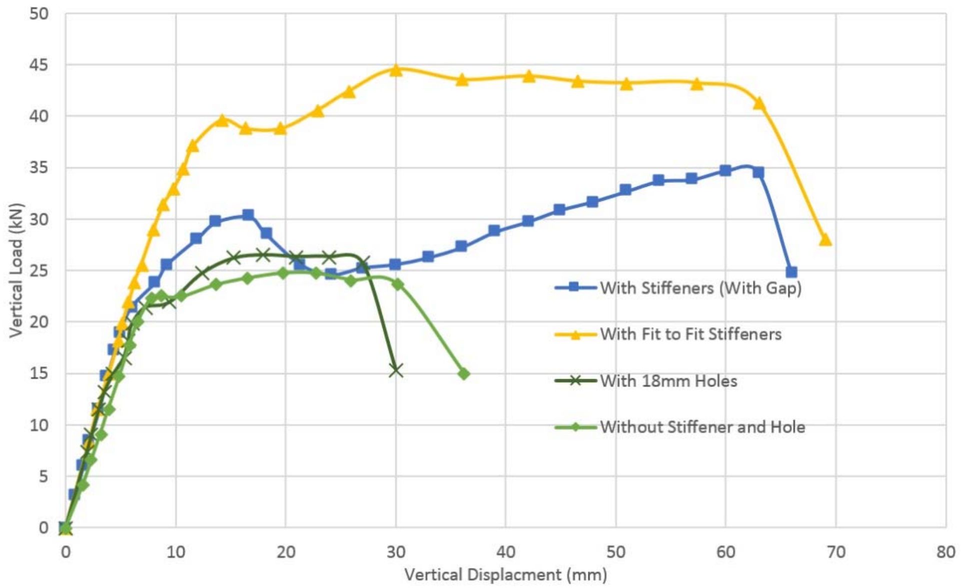


Figure 6: Load versus vertical displacement for SC 100-10 specimens.

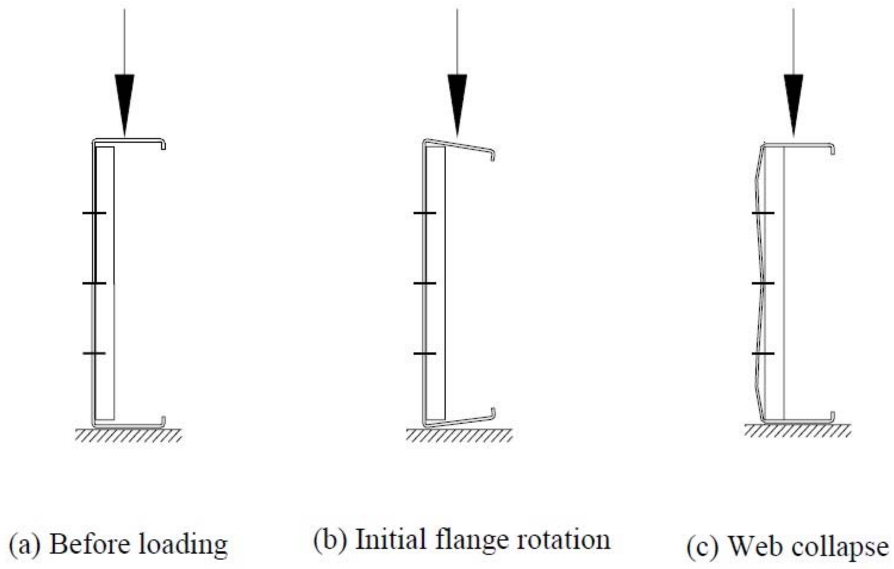


Figure 7: Stiffener Deformation Stages.

3 FINITE ELEMENT MODELING

Finite element (FE) simulation was employed in this study to validate the stress-stain behavior of a cold-formed sections structure under three point bending test. Being very thin along with high width to thickness ratios are among the major characteristics of cold-formed steel sections to observe the effect of localized buckling through the thickness of elements. The three dimensional hexahedral solid (C3D8R) element was assigned to the cold-formed sections model. In order to achieve more accurate results compared to experimental ones, a mesh convergence study in elastic region has been done and an appropriate number of elements were used in simulations. To avoid distortion in the lateral bracing location, a partition was used and this region was constrained along lateral direction to transfer the reaction force to the failure zone. The material properties were extracted from the tensile test and after converting, the true stress- strain values were employed as a piece of wise linear curve to ABAQUS/STANARD. The failure criterion was assumed as a specific fracture strain that was calculated from the monotonic tensile test. Surface to surface contact was used between rigid supports and the test specimens; however, to model the self-drilling screw, the tie algorithm has been applied in the drilling affected zone that would create the same condition with test experiments. The failure modes predicted by the FE analysis were compared with the test results for one specimen, as shown in Figure 8. The failure modes of local buckling (specimens with web bearing stiffeners) and distortional buckling (specimens without stiffeners) obtained from the FEA at ultimate load were in good agreement with test results.

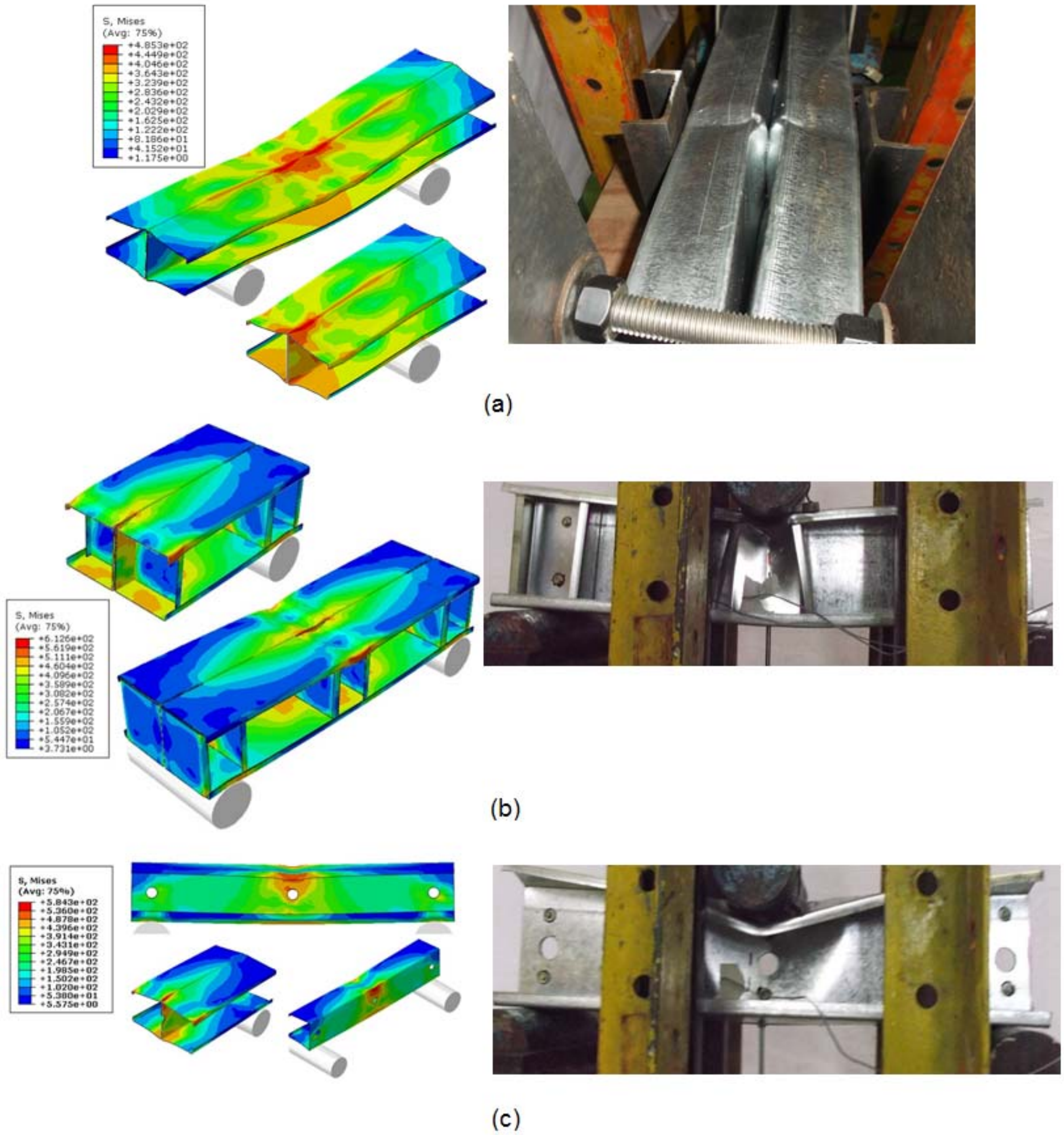


Figure 8: A comparison between experimental and numerical failure modes (a) specimen without stiffeners and hole, (b) specimen with web bearing stiffeners and (c) specimens with web circular holes.

4 INTERACTION RELATION FOR BENDING AND SHEAR

The existing Direct Strength Method used in this study to calculate the local and distortional buckling in accordance with North American Specification (2012) as briefly introduced in the following paragraphs: the nominal flexural strength (Mn), which is the smaller value of the nominal Flexural strength used as distortional buckling (Msd) and nominal flexural strength used for local buckling (Msl), is given in Eq. (3)

$$M_n = \min(M_{sl}, M_{sd}) \tag{3}$$

Since installation of lateral bracing results in no torsional buckling, the specimens could be treated as fully braced beams. Therefore, referring to Appendix 1.2.2.1 of the North American Specification AISI 2007, the yield moment (My) is used instead of the nominal flexural strength (Mn) for lateral-torsional buckling (Mse) in fully braced beams.

The nominal member moment capacity at local buckling (M_{sl}) will be calculated from Appendix 1, Section 1.2.2.2 of AISI 2007 (2012):

$$For \qquad \lambda_l \leq 0.776: \qquad M_{sl} = M_{se} \tag{4}$$

$$For \qquad \lambda_l > 0.776: \qquad M_{sl} = \left[1 - 0.15\left(\frac{M_{ol}}{M_{se}}\right)^{0.4}\right]\left(\frac{M_{ol}}{M_{se}}\right)^{0.4} M_{se} \tag{5}$$

where,

λ_l = non-dimensional slenderness incorporated to calculate $M_{sl} - \lambda_l = \sqrt{M_{se}/M_{ol}}$; M_{se} = nominal moment capacity of the member regarding the lateral torsional buckling of the full section; M_{ol} = elastic local buckling moment of the section: $M_{ol} = Z_f f_{ol}$; f_{ol} = the section's elastic local buckling stress in bending; Z_f = section modulus about a horizontal axis of the full section. As mentioned before, lateral braces have been used in this study to prevent the lateral-torsional buckling so that $M_{se} = M_y$ in Eq. (5), where $M_y = Z_f f_y$.

Appendix 1, Section 1.2.2.3 of AISI 2007 (2012) is used to determine the nominal moment capacity of the member at distortional buckling (Msd):

$$For \qquad \lambda_d \leq 0.673: \qquad M_{sd} = M_y \tag{6}$$

$$For \qquad \lambda_d > 0.673: \qquad M_{sd} = \left[1 - 0.22\left(\frac{M_{od}}{M_y}\right)^{0.5}\right]\left(\frac{M_{od}}{M_y}\right)^{0.5} M_y \tag{7}$$

where,

λ_d = non-dimensional slenderness incorporated to calculate $M_{sd} - \lambda_d = \sqrt{M_y/M_{od}}$; M_{od} = the section's elastic distortional buckling moment - $M_{od} = Z_f f_{od}$; f_{od} = the section's elastic distortional buckling stress in bending; Z_f = section modulus of the full section about a horizontal axis; and finally $M_y = Z_f f_y$.

The nominal shear strength, Vn, of specimens were determined in conformity with Section C3.2.1 of AISI 2007 as shown in follow:

$$\text{For} \qquad h/t \leq \sqrt{EK_v/F_y} \qquad V_n = A_w 0.6F_y \tag{8}$$

For $\sqrt{EK_v/F_y} < h/t \leq 1.51\sqrt{EK_v/F_y}$ $\quad V_n = A_w \times \left\{0.6\sqrt{EK_vF_y}/(h/t)\right\}$ (9)

For $1.51\sqrt{EK_v/F_y} < h/t$ $\quad V_n = A_w \times \left\{\pi^2 EK_v/12(1-\mu^2)(h/t)^2\right\}$ (10)

where

Vn = nominal shear strength
Aw= area of web element (h×t) where
h= depth of flat portion of web
t=web thickness
E= modulus of elasticity of steel
Kv= shear buckling coefficient (for unreinforced web, Kv= 5.34)
μ= Poisson's ratio for steel = 0.3

The shear buckling coefficient for web with transverse stiffeners is calculated as follows based on section C3.2.1 of AISI 2007:

For $a/h \leq 1$ $\quad K_v = 4 + 5.34/\left(a/h\right)^2$ (11)

For $a/h > 1$ $\quad K_v = 5.34 + 4/\left(a/h\right)^2$ (12)

Where,

a= shear panel length of unreinforced web element (clear distance between bearing web stiffeners, 300mm)

For C-section webs with circular holes, the shear strength shall be calculated in accordance with Eq 8 to 10, multiplied by the reduction factor qs as defined with following equation:

For $c/t \geq 54$ $\quad q_s = 1.00$ (13)

For $5 \leq c/t < 54$ $\quad q_s = c/(54t)$ (14)

Where

$$c = \frac{h}{2} - d_h/2.83$$

The resistance factor (ϕv) of 0.95 shall be used to determine the design shear strength in accordance with LRFD method. Table 3 shows the flexural and shear strengths based on AISI 2007 for SC150-16 and SC100-10 lipped C-channel sections.

Specimen	Pu (kN)	Mt (kN.m)	Vt (kN)	Vn (kN.m)	Msl (kN.m)	Msd (kN.m)	Mp (kN.m)	$\frac{M_t}{M_{sl}}$	$\frac{M_t}{M_{sd}}$	$\frac{M_t}{M_p}$	$\frac{V_t}{V_n}$
SC150-16											
Without Web Stiffener and hole	66.50	14.13	33.25	50.40	19	15.8	24.84	0.74	0.89	0.56	0.70
With Web Stiffeners (with gap)	67.65	14.37	33.82	60.03	19	15.8	24.84	0.75	0.90	0.57	0.60
With Fit to Fit Stiffeners	101.2 0	21.50	50.60	60.03	19	15.8	24.84	1.13	1.36	0.86	0.88
With 28mm Web holes	66.00	14.02	33.00	38.00	19	15.8	24.84	0.73	0.88	0.56	0.91
SC100-10											
Without Web Stiffener and hole	25.50	5.41	12.75	20.00	6.22	5.05	9.41	0.86	1.07	0.57	0.64
With Web Stiffeners (with gap)	34.48	7.32	17.24	21.65	6.22	5.05	9.41	1.17	1.44	0.77	0.80
With Fit to Fit Stiffeners	44.55	9.46	22.27	21.65	6.22	5.05	9.41	1.52	1.87	1.00	1.02
With 18mm Web holes	26.50	5.63	13.25	16.00	6.22	5.05	9.41	0.90	1.11	0.60	0.82

Table 3: Demand to capacities ratio for primarily shear condition tests of SC150-16 and SC100-10 lipped C-channel sections.

In order to evaluate the adequacy of the existing M-V interaction equation suggested by the North American Specification (2012) for sections with web stiffeners and holes, three different cases (A,B and C) were considered. The horizontal axes in cases A, B and C are constant which represent Vt / Vn while vertical axes represent Mt / Msl , Mt / Msd and Mt / Mp , respectively.

Figure 9 reveals that the M-V interaction curve is not in good agreement (for sections with web bearing stiffeners) with the experimental test by using local buckling strength (Msl). This conflict can be explained by the fact that the shear strength, Vn, suggested by North American Specification was determined through using the elastic buckling strength of the shear panel zone that could only predict very conservative results. Moreover, for specimens with fit to fit web bearing stiffeners the load is transferred into the stiffener directly and immediately after slight rotation of joist flanges. Since the bearing stiffener subject to axial compression is much stiffer than the C-section subject to a two-flange-loading, the majority of the load will be carried by the stiffeners. This assumption appears to be borne out by very conservative results for specimens with web bearing stiffeners. Nevertheless, Eq. 1 predicted the behavior of specimens without web stiffeners and holes and specimens with web holes with acceptable tolerance.

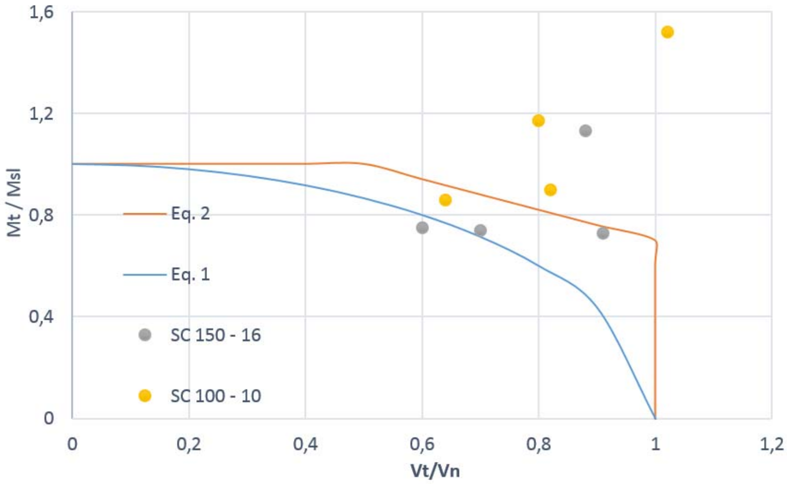


Figure 8: Interaction equation between (Mt /Msl) and (Vt /Vn) (case A).

Figure 10 reveals that Eq.1 provides a better prediction for specimens without web stiffeners and holes by using distortional buckling strength (M_{sd}). However, Eq. 2 established more conservative results for specimens with web bearing stiffeners and holes compare to those of Figure 9. One hypothesis about the behavior of a C-sections with web bearing stiffeners is that, the capacity of the stiffened assembly is the addition of the web crippling capacity of the joist web and the axial capacity of the stiffener. Due to the restraint provided to the joist web by the stiffener, it can be argued that the web crippling of the stiffened joist behaves more like a built-up section than a single web member. Accordingly, the web bearing stiffeners enhance the tension field action and therefore, specimens are enforced to experience the plastic failure mode. All in all, incorporation of the distortional buckling moment into Eq. 1 gives more accurate predictions of M-V interaction behavior for specimens without web stiffeners and holes.

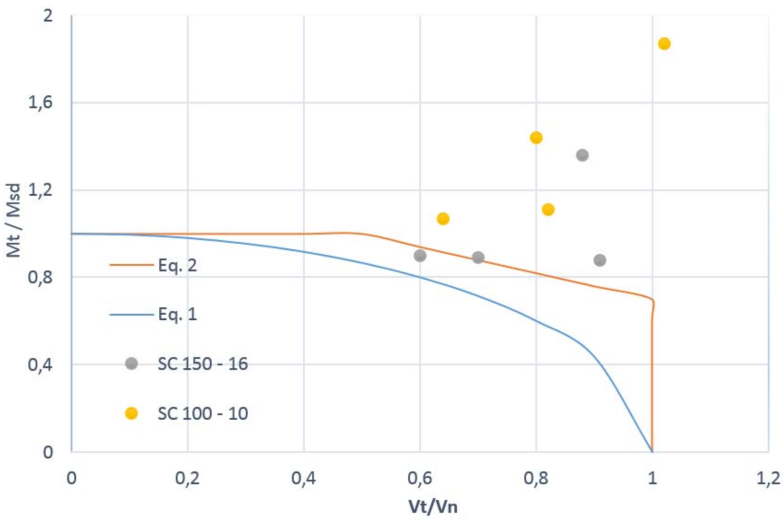


Figure 9: Interaction equation between (Mt /Msd) and (Vt /Vn) (case B).

Figure 11 shows the M-V interaction where the plastic moment Mp was employed to calculate the interaction equation. The curve indicates that Eq. 2 establishes good predictions for specimens with fit to fit stiffeners, especially for SC 150-100 section. This is because plastic moment, Mp, is higher than Msl and Msd and accordingly, using the plastic moment, Mp, for this sections that experienced the plastic failure mode resulted in more accurate predictions. On the other hand, Eqs.1 is un-conservative for specimens without holes and stiffeners and specimens with web holes. This issue can be explained by the fact that this specimens experienced failure as a result of elastic distortional or local buckling before developing the full plastic moment.

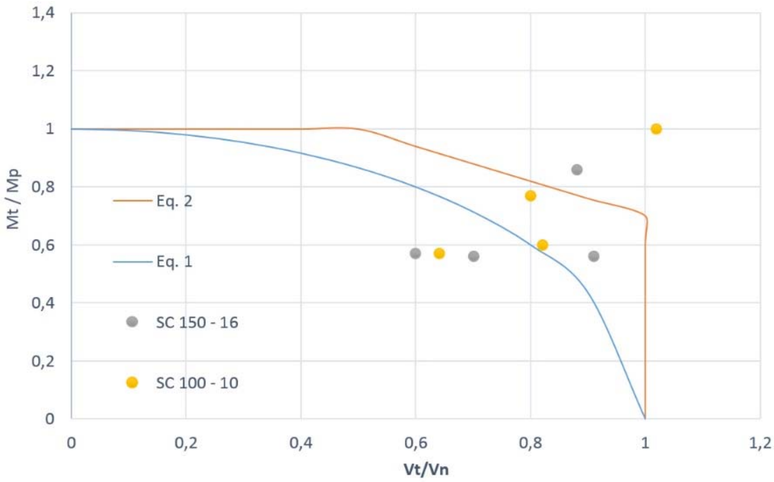


Figure 10: Interaction equation between (Mt /Mp) and (Vt /Vn) (case C).

5 CONCLUSION

In this research, experimental and analytical investigations were carried out to determine the adequacy of existing M-V interaction equation prescribed by North American Specification (2012) for specimens with web baring stiffeners and circular holes subjected to primarily shear condition tests. A total of eight specimens were tested at the Laboratory of Structures and Materials, Universiti Teknologi Malaysia (UTM). The results explicitly indicated that utilizing web bearing stiffeners in cold-formed C sections would reduce non-dimensional slenderness that would eventually improve buckling capacities. Besides, it was concluded that the existing DSM prediction formulas predicted conservative results for specimens with fit to fit cover plate as they can develop the full plastic moment. Accordingly, an alternative DSM formula was proposed that used plastic moment, Mp, rather than the yield moment, My, that provided precise predictions regarding the post-buckling strength of sections with fit to fit stiffeners in case of primarily shear condition test. However, the proposed M-V interaction equation resulted in un-conservative results for slender sections as they experienced elastic distortional or local buckling before developing the full plastic moment. The test results also proved that using distortional buckling strength (Msd) into existing M-V interaction equation predicted reasonable results for specimens without web stiffeners and holes while using local buckling strength (Msl) recommend into M-V interaction equation when the sections possess web circular holes.

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