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Local buckling of I-shape members bent about their weak axis

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Abstract

Steel design standards in North America and Europe provide plate slenderness limits for the web and flanges to prevent local buckling of I-shapes before they attain the desired stress level. While significant research has been conducted on local buckling of I-shapes under strong axis bending, very little has been done on local buckling of I-shapes when they bend about their weak axis with the action of a superimposed axial force. Beam bending about weak axis is common in many framed structures. In the current editions of ANSI/AISC 360-16 and CSA S16-14, slenderness limits are provided to check local buckling of I-shapes when subjected to weak axis bending. One important difference between AISC and CSA local buckling requirements for I-shapes is that while, for any certain compactness, AISC recommends same slenderness limits of for both strong and weak axis bending, in Canadian standard, for class 3 section, flange slenderness requirements are different between strong and weak axis bending. The slenderness limits for weak axis bending in S16 have never been assessed experimentally or numerically. Strain distribution in I-shapes when bent about the strong-axis is significantly different from the strain distribution when bent about weak axis. Thus, the use of same slenderness limits for both strong-axis and weak-axis bending conditions is questionable, and should be investigated. This paper evaluates the slenderness limits in the current North American design standards for local buckling of I-shapes bent about the minor axis, with and without axial load. A nonlinear finite element (FE) model is developed using the commercial finite element software ABAQUS. A series of FE analysis considering various parameters such as web slenderness ratio (h /w), flange slenderness ratio (b /2t) is conducted to evaluate the current slenderness limits.

1. Introduction

I-shaped (known as W shapes in Canada and many other countries) steel sections are commonly used in steel buildings and bridges. They are often used as beams for transferring bending moments; as columns for transferring axial compression and as beam-columns for transferring combined axial compression and bending. When the beam or beam-column is subjected to applied external load, it is possible that at some level of compressive stress the thin plate elements that make up the cross-section (i.e., flanges and web) will buckle. Once the section buckles, no more loads can be applied. Thus, local buckling of component plates of W shapes is one limit which must be met in limit state design of steel beam and beam-columns. Figure 1

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shows a beam segment with the compression flange already buckled. The compression flange is assumed to be braced to prevent any lateral deflections of the flange. The web plate is assumed to be stocky enough to prevent vertical buckling of the flange. Thus, as shown in Fig. 1, the local buckling mode consists of a twisting motion of the flange, together with a rotation of the web plate.

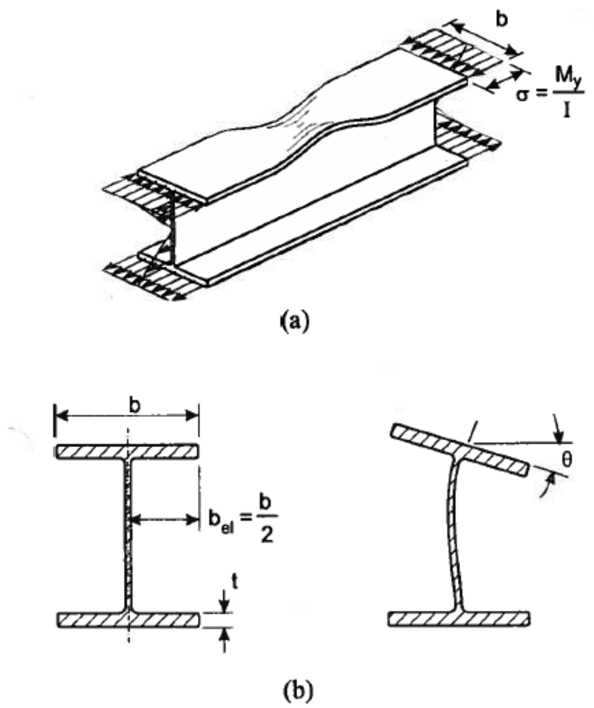


Figure 1: Typical local buckling mode of W-shape member (Kulak and Grondin 2018)

It is well known that stability of a plate is mostly affected by its width- to-thickness ratio. Other factors which affect plate stability in W-shapes are the plate edge support conditions, the type of stress distribution on the cross-section of the member, and the material properties, more specifically yield strength of the steel. Thus, current design standards around the world, such as CAN-CSA S16-14 (CSA 2014), AISC-ANSI 360-16 (AISC 2016), and EN1993-1-1 (Eurocode 3)], specifies limitations on the width-to-thickness ratios of webs and flanges in terms of square root of the yield stress of the material used. Depending on the width-to-thickness ratios of their flanges, W-shapes are classified as Class 1, Class 2, Class 3, and Class 4 sections. Class 1 sections (plastic design) have the capacity to attain their moment capacity (M_p) and undergo large inelastic rotations. Class 2 section (compact section) corresponds to one which can reach its plastic moment capacity but with limited rotation capacity. Thus, limiting width-to-thickness ratio for class 2 section is less restrictive than that for the class 1 section. A class 3 section (non-compact) beam or beam-column is not able to reach the plastic moment capacity, but is able to reach or exceed the yield moment capacity (M_y) before local buckling. Sections which are too slender to meet the requirements for Class 3 sections are classified as Class 4 sections. Class 4 sections are light-gauge cold-formed sections and are not considered in this research. Table 1 presents the limiting width- to-thickness ratios for different W-sections subjected to both strong and weak axis bending.

Table 1: CSA S16-14 requirements for local buckling of beam and beam-column

Element	Section classification		
	Class 1	Class 2	Class 3
Axial compression and bending about the strong (major) axis			
Flanges	$\frac{b_{el}}{t} \leq \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}$
Webs	$\frac{h}{w} \leq \frac{1100}{\sqrt{F_y}}\left(1 - 0.39\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \leq \frac{1700}{\sqrt{F_y}}\left(1 - 0.61\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \leq \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Axial compression and bending about the weak (minor) axis			
Flanges	$\frac{b_{el}}{t} \leq \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{340}{\sqrt{F_y}}$
Webs, when $C_f > 0.4\phi C_y$	$\frac{h}{w} \leq \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{525}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$
Webs, when $C_f \leq 0.4\phi C_y$	$\frac{h}{w} \leq \frac{1100}{\sqrt{F_y}}\left(1 - 1.31\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \leq \frac{1700}{\sqrt{F_y}}\left(1 - 1.73\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \leq \frac{1900}{\sqrt{F_y}}\left(1 - 0.65\frac{C_f}{\phi C_y}\right)$

American steel standard AISC-ANSI 360-16 provides similar width- to-thickness limitations, presented in Table 2, for compact (equivalent to Class 2 section), non-compact (equivalent to Class 3 section), and slender section (equivalent to Class 4 section).

Table 2: AISC 2016 requirements for width-to-thickness ratios for flexural member

Element	Section classification	
	Compact	Non-compact
Bending about the strong (major) axis		
Flanges of rolled I-shapes	$\frac{b_{el}}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$	$\frac{b_{el}}{t} \leq 1.0\sqrt{\frac{E}{F_y}}$
Flanges of doubly symmetric built-up I sections	$\frac{b_{el}}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$	$\frac{b_{el}}{t} \leq 0.95\sqrt{\frac{k_c E}{F_L}}$
Webs of doubly symmetric I sections	$\frac{h}{w} \leq 3.76\sqrt{\frac{E}{F_y}}$	$\frac{h}{w} \leq 5.70\sqrt{\frac{E}{F_y}}$
Bending about the weak (minor) axis		
Flanges of all I-shapes	$\frac{b_{el}}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$	$\frac{b_{el}}{t} \leq 1.0\sqrt{\frac{E}{F_y}}$
Webs of doubly symmetric I sections	$\frac{h}{w} \leq 3.76\sqrt{\frac{E}{F_y}}$	$\frac{h}{w} \leq 5.70\sqrt{\frac{E}{F_y}}$
Note: for the non-compact built-up section flange, $k_c = \frac{4}{\sqrt{\frac{h}{w}}}$ with $0.35 \leq k_c \leq 0.76$ $F_L = 0.7F_y$ for built-up I-shaped members under strong axis bending with compact and non-compact web and $\frac{S_{xt}}{S_{xc}} \geq 0.7$; when $\frac{S_{xt}}{S_{xc}} < 0.7$, $F_L = F_y\frac{S_{xt}}{S_{xc}} \geq 0.5F_y$ for built-up I-shaped members under strong axis bending with compact and non-compact web		

Early research on buckling of web and flange of I-shape members were based on several simplifying assumptions. Haaijer and Thurlimann (1959) conducted an experimental and analytical investigation to determine maximum plate width-to-thickness ratios for W shapes suitable for plastic design. The experimental investigation of W shapes included six axial specimens and six flexural specimens. For analytical solutions for buckling of web or flange, plates with either simple support or fully rigid support at the web-to-flange junctions were assumed. Although Haaijer and Thurilmann (1959) did not test any specimens subjected to axial and flexural loadings combined, they used the results of a semi-empirical method to suggest plate width-to-thickness values for such members. Code limitations, both in CISC and AISC, for flanges and webs of W shapes were developed based on the study of Haaijer and Thurlimann (1959).

Kulak and his research team at the University of Alberta (Holtz and Kulak 1973; Nash and Kulak 1976; Perlynn and Kulak 1974, Dawe 1980) conducted series of experimental and analytical studies to investigate the width-to-thickness limitations of Canadian standard for W-shapes subjected to strong axis bending with and without axial compressive force. As a result of their works, changes in the web limitation requirements for W shapes were implemented for Class 2 and Class 3 sections.

While significant research has been done on local buckling of I-shapes under strong axis bending, very little has been done on local buckling of I-shapes subjected to weak-axis bending. Although we can specify the same slenderness limits for the flanges when bent about their weak axis as we use for bending about the strong axis, we know that under a strain gradient, such as exists for bending about the weak axis, flange plates will not buckle as readily as flange plates subjected to a uniform strain distribution. Similarly, since the web is subjected to lower strains over its full height when subjected to bending about the weak axis, its resistance to local buckling is expected to be better than the web of an I-shape member bent about its strong axis.

The primary objective of this paper is to assess the slenderness limits provided in the Canadian design standard for local buckling for beam and beam-columns. To achieve this objective a detailed FE analysis is carried out for simply supported I-shape beams and beam-columns subjected to weak-axis bending with and without axial compressive force. The FE model developed is initially validated against tests conducted at the University of Alberta on beam and beam-columns under strong-axis bending.

2. Finite Element Model for Local Buckling of W-shapes

To simulate the local buckling behavior of a steel beam or beam-column when subjected to strong and weak axis bending with and without compressive axial force, a nonlinear finite element (FE) model is developed using the commercial finite element software package ABAQUS (ABAQUS 2013). The FE model included both geometric and material nonlinearities.

2.1 Elements and Mesh Configuration

Shell element is most widely used for complex buckling behaviour because of its capability of providing accurate solutions in case of a structure whose thickness is much smaller than the other dimensions (Smalberger 2014). In this study, 4-node doubly curved shell with reduced integration (ABAQUS element S4R) has been chosen to model the web and flanges of the W sections. Element type S4R accounts for finite membrane strains and arbitrary large rotations, which makes them suitable for large-strain analysis (ABAQUS 2013).

For the FE model, 8 elements across the width of flange and 20 elements along the height of web, as shown in Fig. 2, was used. This configuration of the mesh was obtained from a mesh sensitivity analysis. A typical value of modulus of elasticity, $E = 200{,}000\, MPa$, nominal yield stress, $F_y = 350\, MPa$ and Poison's ratio of 0.3 was used in this study. A bilinear elastoplastic stress versus strain curve is assumed for all the models. In addition, a nonlinear isotropic strain hardening of 1% of the elastic stiffness was considered for all analyses

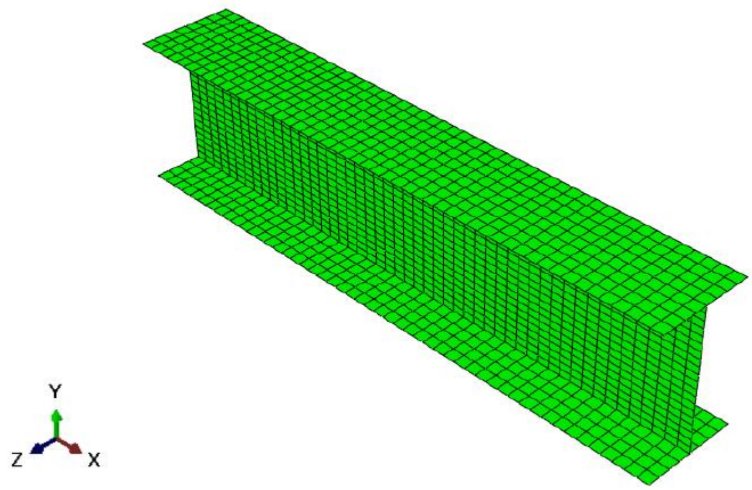


Figure 2: FE Mesh for configuration of finite element model

2.2 Boundary Conditions

The beams considered in this study are simply supported at both ends. Idealised simply supported boundary conditions that allow for major and minor axis rotations and warping displacements while preventing in-plane and out-of-plane deflections and twists, were used at the supported end of the member. These boundary conditions have been incorporated into the FE model by the following criteria:

1. Simply supported in-plane: The centroids at both ends were restrained against in-plane y-axis deflection $(U_y = 0)$ but unrestrained against major and minor axis rotations $(\phi_z \neq \phi_y \neq 0)$. Also, one end was restrained against z-axis displacement $(U_x = 0)$.

2. Simply supported out-of-plane: All web nodes including the centroid of both ends were restrained against out-of-plane z-axis deflection $(U_z = 0)$. Also, both ends were restrained against twist $(\phi_x = 0)$, but unrestrained against minor axis rotation and flange warping displacement.

2.3 Analysis Type

Two types of analysis i.e. elastic buckling analysis and non-linear static analysis are conducted in this study. First, an eigenvalue analysis is performed for elastic buckling analysis in which eigenvalues of corresponding eigenmodes are obtained from linear perturbation buckling analysis. The appropriate eigenmode represents the imperfection for the non-linear static analysis. A RIKS analysis is selected to do the nonlinear post buckling analysis since this technique is usually suitable for predicting the instability as well as for understanding the non-liner behavior of geometric collapse (ABAQUS 2013).

2.4 Geometric Imperfection and Residual Stress

Geometric imperfections are important and should be included in stability analysis. Since this study is about local buckling of W-shapes, global imperfection such as out-of-straightness is not considered and only local imperfections are taken into consideration. Currently, in the absence of any universal guidelines, engeneers prefer to use manufacturing tolerances (ASTM A6/A6M-04b 2003). Another practice is to introduce an initial web out of flatness of d/150 and/or an initial tilt in the compression flanges of b_f /150 (Kim and Lee 2002). In this study, first buckling mode from eigenvalue buckling analysis is obtained. The buckling mode shape is then scaled with either web out-of-flatness (d/150) or tilt in the compression flange (b_f/150), whichever produces higher imperfection.

Results from different experiments showed that residual stress can be dependent on few parameters such as manufacturing processes, geometry of the section, fabrication process etc. (McFalls and Tall 1969; Alpsten and Tall 1970). For this study, a commonly used residual stress pattern, as suggested by Chernenko and Kennedy (1991) and shown in Fig. 3, with a maximum compressive stress of 30% of yield stress (0.3Fy) at the flange tips and maximum tensile stresses (0.3Fy) at web-to-flange junctions and web center is used In the FE model, the residual stresses are specified directly using the predefined field feature of ABAQUS as initial stresses, and given as the average value across the element at its center. A static step is also defined prior to RIKS analysis for the equilibrium of residual stress. No load is applied during this step.

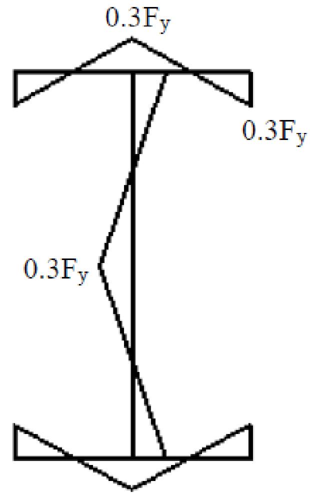


Figure 3: Typical residual stress pattern (Chernenko and Kennedy 1991)

2.5 *Validation of FE model with experiments*

The FE model developed is validated against the test results of twelve beam specimens (Holtz and Kulak 1973, 1975) and nine beam-column specimens (Perlynn and Kulak 1974) subjected to strong-axis bending. All the specimens were fabricated from CSA G40.12 plate, which had a specified minimum yield strength of 44 ksi (303 MPa). The corresponding initial imperfection for each specimen, as reported in the tests, was incorporated in the FE analysis. Since no information of residual stresses was available, the commonly used residual stress pattern as shown in Fig. 3 was assumed for all analysis.

Holtz and Kulak (1973) tested ten compact beam specimens. All the beam specimens were simply supported and loaded symmetrically with equal concentrated loads acted on compression flanges so that a uniform moment region existed between load points. All beams were laterally braced at the load and reaction points to prevent any lateral buckling. Holtz and Kulak (1975) also tested two non-compact beams. In table 3, the first ten specimens are compact beams and the last two beams (WS-12-N and WS-13-N) are non-compact beams. The flanges of eight compact specimens were 184.15 mm by 9.53 mm, resulting in a width to thickness ratio of 9.66. The flanges of specimens WS-7-P and WS-8-P were 152.40 mm by 9.53 mm. For the two non-compact beams, the flanges were 279.40 mm by 9.53 mm. The webs of all specimens were 6.35 mm thick. Finite element analyses of all the beam specimens were carried out. Moments obtained from FE analysis are compared with the experimental values in Table 3. A very good agreement is observed between experimental and predicted buckling moments for the specimens tested by Holtz and Kulak (1973, 1975). As shown in Table 3, the mean test to predicted moment capacity ratio is 1.002 and the coefficient of variation is 0.04.

Perlynn and Kulak (1974) tested nine compact beam-columns subjected to strong axis bending and axial compressive force. The flanges of all specimens were 184.15 mm by 9.53 mm and the webs of all specimens were 6.35 mm thick. The web depth (h) of each specimen was varied and is reported in Table 4. The specimens all had a clear height of 1143 mm. Each specimen was aligned in a universal testing machine, which was used to apply the axial compressive force. A moment was superimposed by applying an eccentric load at both ends. The eccentric load was increased to provide an increment of moment and the principal load was decreased so that the total axial load remained constant. All the specimens were laterally braced at mid-span and adequate torsional restraint was provided at the ends by means of the rigidly connected loading arms. The critical buckling moments for the compact beam-columns tested by Perlynn and Kulak (1974) are presented in Table 4. The ratios of the applied axial load to the yield load $\left(\alpha = \frac{C_f}{C_y}\right)$ are also shown for each specimen. The predicted value of the local buckling moment from FE analysis as well as the ratio of predicted to experimental moment is also shown for each specimen. It is observed from Table 4 that the developed FE model can predict the test results reasonably well. The mean test to predicted moment ratio is 1.03 and the coefficient of variation is 0.04. Figure 4 shows typical FE meshes of two specimens, PK-4 of Perlynn and Kulak (1974)-left and WS-3 of Holtz and Kulak (1973)- right, when local buckling was overserved

Table 3. Comparison of FE analysis results with test results of Holtz and Kulak (1973, 1975)

Specimen	Span (mm)	Distance of applied load from end (mm)	Moment at buckling (kN·m)		Ratio (Test/FE)
			Experiment	FE Analysis	
WS-1	4876.8	1828.8	434	444	0.98
WS-2	4879.3	1828.8	555	549	1.01
WS-3	6096.0	2514.6	649	636	1.02
WS-4	7307.6	3195.3	771	745	1.03
WS-6	4879.3	1828.8	445	455	0.98
WS-7-P	4264.7	1673.9	382	398	0.96
WS-8-P	4267.2	1676.4	426	448	0.95
WS-9	3048.0	1066.8	442	432	1.02
WS-10	3045.5	1115.1	517	474	1.09
WS-11	3042.9	1216.7	540	511	1.06
WS-12-N	4876.8	1981.2	606	634	0.96
WS-13-N	5486.4	2286.0	644	663	0.97

Table 4 Comparison of FE analysis results with test results of Perlynn and Kulak (1974)

Specimen	Web depth (mm)	C_f/C_y	Moment at buckling (kN·m)		Ratio (Test/FE)
			Experiment	FE Analysis	
PK-1	307.1	0.2	268	256	1.05
PK-2	341.1	0.2	309	291	1.06
PK-3	401.8	0.2	326	323	1.01
PK-4	260.9	0.4	181	181	1.00
PK-5	295.7	0.4	207	200	1.03
PK-6	354.8	0.4	260	236	1.10
PK-7	261.1	0.8	83	79	1.05
PK-8	295.9	0.8	78	73	1.08
PK-9	352.6	0.8	66	68	0.96

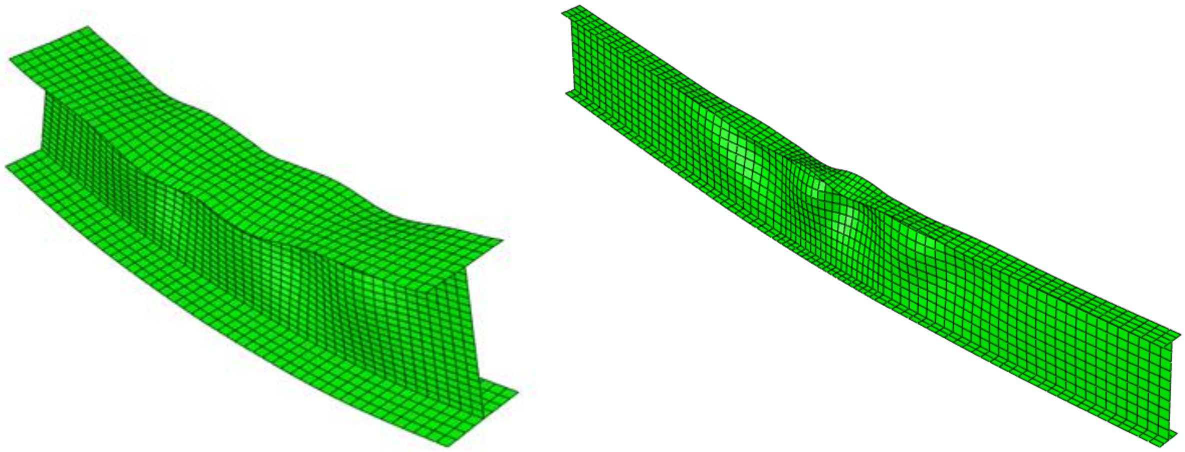
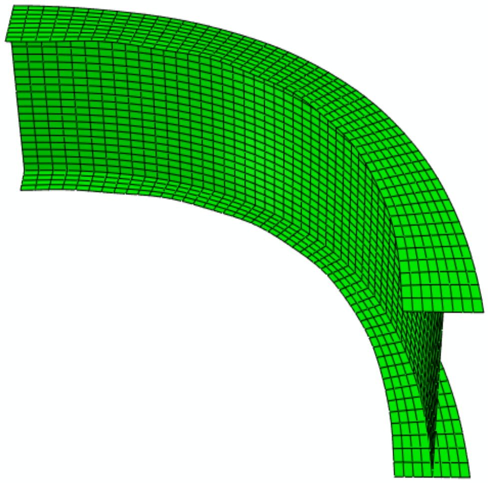
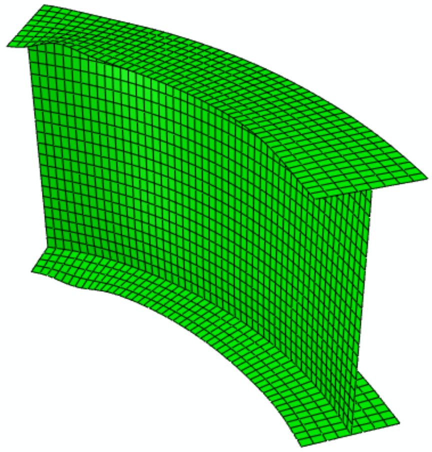


Figure 4: State of specimens at local buckling (FE model PK-4 left and FE model WS-3 right)

3. Local buckling of W-shape beams subjected to weak-axis bending

With the validated FE model two W-shape beams, W250x73 and W530x74, were analysed for both strong-axis and weak-axis bending. A uniform constant moment along the length was applied for both beam sections. Assuming yield strength of 350 MPa, the unbraced lengths to preclude lateral torsional buckling of both beam sections are 4010 mm and 2040 mm, respectively. A span length of 2000 mm is assumed for both beam sections. As per the CSA S16-14, W250x73 beam is class 2 section with class 2 flanges and W530x74 beam is class 1 section. For the FE analysis, residual stress pattern shown in Fig. 3 is considered. For this study, a strain hardening of 2 percent of the elastic stiffness is considered for all analysis. The von Mises yield criterion was adopted for the all the analyses. FE analysis shows that for the strong axis bending, both the beams were able to reach plastic moment followed by local buckling in compression flange. The ratio $M_{FE/}M_P$ for W530x74 and W250x73 were obtained as 1.11 and 1.05 respectively. When both beams were subjected to weak-axis bending, no local buckling was observed and the beams experienced large out of plane deflections. Even when the length of beam W530x74 is 1000 mm, no local buckling was observed. Figure 5 shows the shape of FE mesh of W530x74 section when subjected to weak-axis bending. Local buckling at the compression flanges was observed for the W530x74 beam when the span length was 800 mm. The deflected shape on the left of Fig. 5 is obtained when out-of-plane deflection is 100 mm and the deflected shape shown at the right of Fig. 5 is for W530x74 beam with a length of 800 mm. Thus, local buckling of W-shape beams bent about weak axis is not important and might not to be considered when the unbraced length for weak-axis bending is equal and larger than the unbraced length for strong-axis bending. Another observation made from these analyses is that web buckling of W-shape beam bent under weak-axis bending is unlikely to occur. In the next section, a parametric study will be conducted to investigate the CSA S16 local buckling requirements of beams bent about their weak axis.





For span of 2000 mm For span of 800 mm

Figure 5: FE mesh of W530x74 section under weak-axis bending for different spans

4. Assessment of local buckling limits of W-shape beams bent about weak-axis

A parametric study was conducted to investigate effect of flange and web slenderness ratios on local buckling of W-shape beams when subjected to weak axis bending. First, two groups of beam sections, each group has five beam sections, with web slenderness ratios of $\frac{h}{w}=\frac{968}{\sqrt{F_y}}$ and $\frac{h}{w}=\frac{1073}{\sqrt{F_y}}$ were analysed for constant moment along the length. The beam sections selected from CISC steel handbook were W530x74 and W610x82. Between each group of beams depth, web thickness and flange thickness of the beams were kept constant and the flange width was varied to get different flange slenderness ratios. For all the analysis the unbraced length was constant. The web slenderness ratios for all the ten (10) beam sections met the class 1 web requirement of CSA S16. Figure 6 shows results of all the ten beam sections for different flange slenderness ratios. It is observed that for all the beam sections that meet the CSA class 1 $\left(\frac{145}{\sqrt{F_y}}\right)$ and class 2 flange $\left(\frac{170}{\sqrt{F_y}}\right)$ limits were able to reach the plastic moment. Figure 6 also shows that the slenderness limit for class 2 flange can be increased to $\frac{200}{\sqrt{F_y}}$. Next, seventeen (17) beam sections that meet CSA class 3 web limit of $\frac{1900}{\sqrt{F_y}}$ were analysed for different flange slenderness ratios. Out of seventeen beams, eleven beams, which were obtained by modifying the flange slenderness ratios of W530x74, had web limit of $\frac{1749}{\sqrt{F_y}}$ and the rest six sections, which were obtained by modifying the flange slenderness ratios of W610x82, had web limit of $\frac{1822}{\sqrt{F_y}}$. Finite element analysis shows that all the beams were able to reach their yield limit. It is also observed from Fig. 7 that the current flange limit of $\frac{340}{\sqrt{F_y}}$ for class 3 beam subjected to weak axis bending is overly conservative and the results show that limit should be increased to a value of $\frac{450}{\sqrt{F_y}}$, which is close to the recommended limit of AISC 360-16.

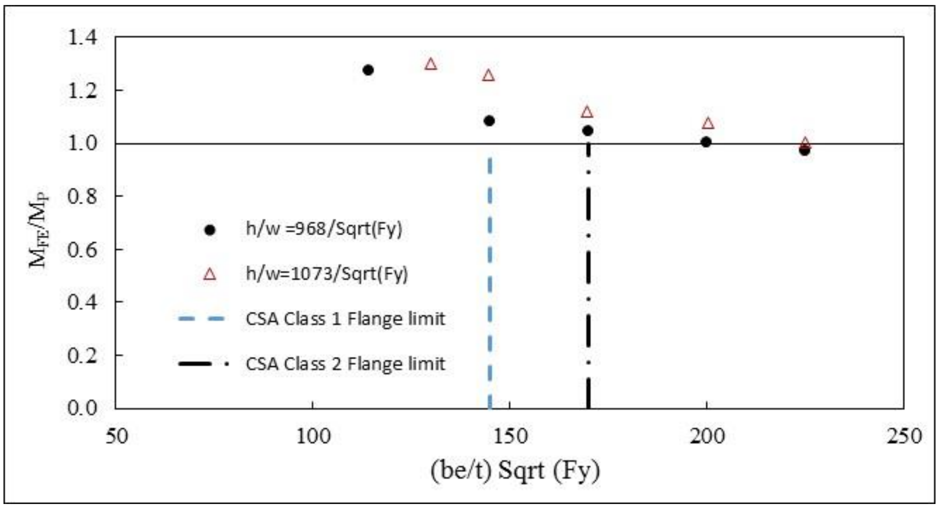


Figure 6: Effect of flange slenderness for beams with Class 1 web

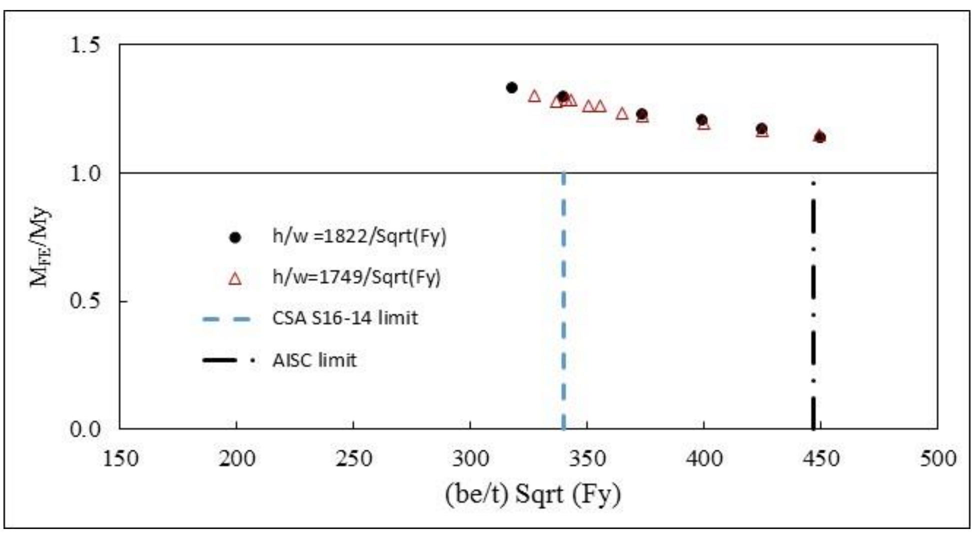


Figure 7: Effect of flange slenderness for beams with Class 3 web

To investigate the effect of web slenderness on moment capacity of W-shape beams under weak-axis vending, seven beams with constant flange slenderness limit of 7.11 that meets the CSA class 1 flange limit were analysed for different web slenderness varying between the class1 web limit $\left(\frac{1100}{\sqrt{F_y}}\right)$ and class 2 web limit $\left(\frac{1700}{\sqrt{F_y}}\right)$. The beam sections were obtained by modifying the web thickness of W1000x222 section, obtained from CISC steel handbook. Results for only five web slenderness ratios are presented in Fig. 8 for clarity. It is observed that all the beams were able to reach the plastic moment capacity followed by large plastic deformation. All the beams showed similar moment deformation behavior. This shows that current CSA web limit for minor axis bending can safely be increased to $\frac{1700}{\sqrt{F_y}}$, which is the recommend web limit for compact section in AISC 360-16.

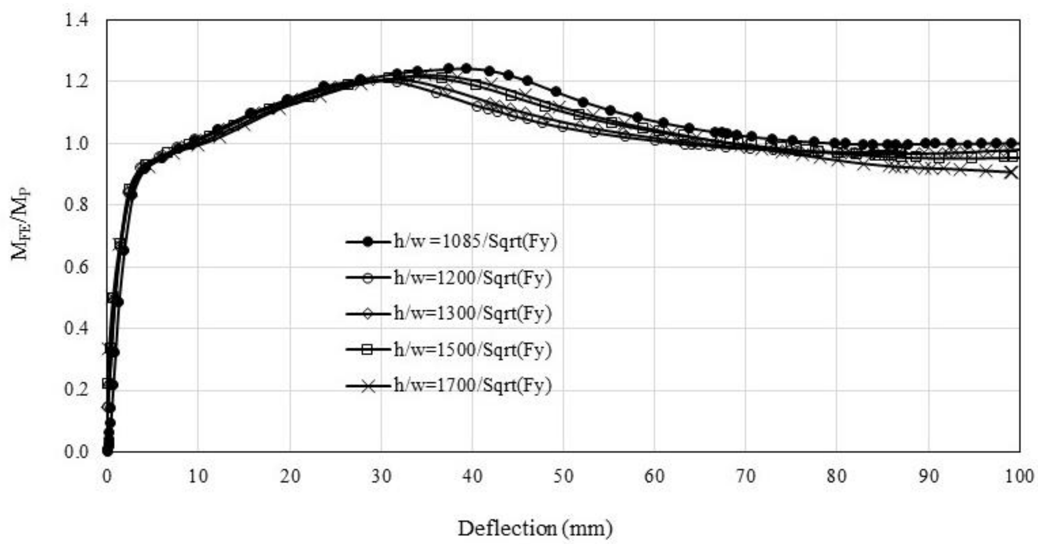


Figure 8: Effect of beam web slenderness on moment capacity with Class 1 flange

In addition, five beams with constant flange slenderness limit of 18.02, which meets CSA class 3 flange limit, were analysed for different web slenderness. The beam sections were obtained by modifying only the web thickness of W1100x343 section. It is observed from Fig. 9 that all the beams were able to reach the yield moment capacity. It is also observed that current CSA web limit for class 3 beam section for minor axis bending is overly conservative. The limit can safely be increased to $\frac{2600}{\sqrt{F_y}}$, which is also close to the recommend web limit for non-compact section in AISC 360-16.

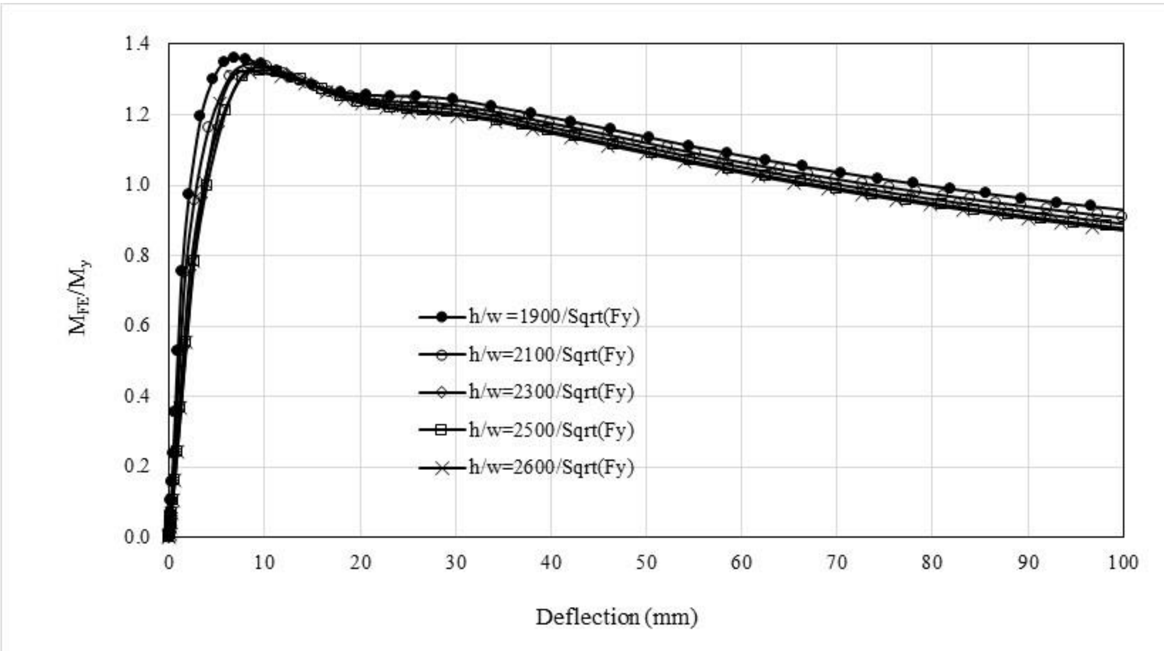


Figure 9: Effect of beam web slenderness on moment capacity with Class 3 flange

5. Assessment of local buckling limits of W-shape beam-columns bent about weak-axis

To date few analyses have been conducted to investigate the CSA web and flange slenderness limits for W-shape beam-columns when bent about their weak axis. Five beam-columns with constant web limit of 43.04, which meets the class 1 web limit of beam-column for minor axis bending, were analysed to see if the flange slenderness limit of Class 1 beam-column can be relaxed. The beam-column sections are obtained by modifying the flange width of W530x74 section. Results with only α value (ratio of the applied axial load to the yield load) of 0.2 are presented in Fig. 10. More analysis with other α values (0.4, 0.5, 0.6 and 0.8) are currently in progress and will be available at a later date. Figure 10 shows that current flange slenderness limits for both class 1 and class 2 beam-column sections are adequate when the beam-column sections are subjected to weak axis bending in addition to a compressive axial force of 20% of the axial load capacity.

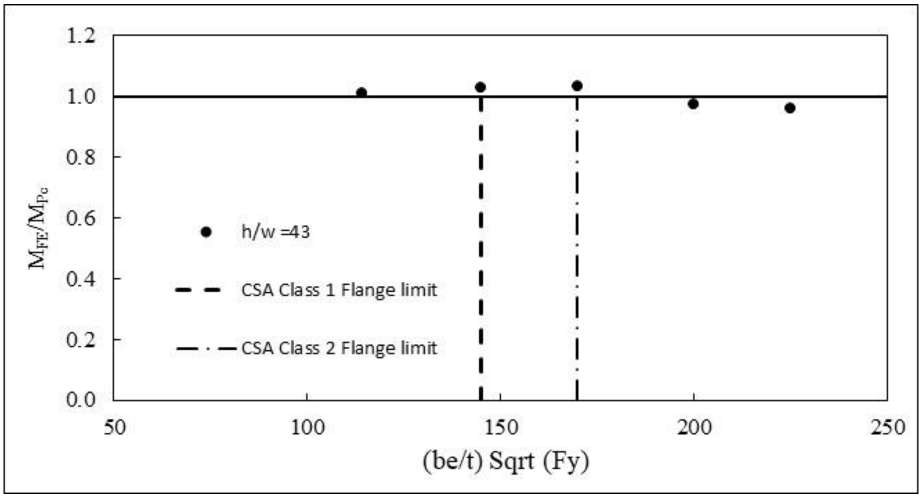


Figure 10: Effect of flange slenderness on moment capacity of beam-columns with Class 1 web

6. Future Work

It is acknowledged that the conclusions obtined in this paper are based on limited number of analysis. Currently work is in progress to extend the parametric study. The available interaction relations for beam-columns bent about their weak-axis will be assessed and, if required, improved interaction equations will be proposed. In addition, effect of different parameters such as imperfections, residual stress, and material stress- strain curve and support conditions on behavior of beams and beam-columns bent about their weak axes will be studied.

7. Conclusions

A nonlinear FE model was developed for studying the local buckling behaiour of I-shape members when subjected to weak axis bending. The FE model was validated against the test results from two test programs on beam and beam-columns. It was observed that the developed FE model can reasonably predict the local buckling behavior of W-shape beam and beam-columns when bent about their strong axis. With the validated FE model, a series of FE analysis was conducted to investigate flange and web slenderness ratios of beam and beam columns in the current Canadian standard. FE analysis showed that local buckling of a W-shape beam bent about weak axis is not important provided the beam satisfies the slenderness requirements for strong axis bending and has a length equal or greater than the maximum unbraced length to preclude lateral torsional buckling. It was also observed that current Canadian flange slenderness limit in for class 3 beam subjected to weak axis bending is overly conservative and should be increased to a value of $\frac{450}{\sqrt{F_y}}$, which is close to the recommended value in AISC 360-16. Study also shows that current CSA web limit for class 3 beam section for minor axis bending is overly conservative. The limit can safely be increased to $\frac{2600}{\sqrt{F_y}}$, which is also close to the recommend web limit for non-compact section in AISC 360-16.

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