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Moment Resistances of wide flange beams with initial imperfection and residual stresses

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ABSTRACT

Elastic and inelastic moment resistances of W-steel beams with considering the effects of initial imperfections and residual stresses are numerically investigated in the present study. The numerical model is implemented in ABAQUS in which residual stresses are incorporated by using initial conditions while the initial imperfection is imported through the first lateral-torsional buckling mode. By comparing the FEA moment resistances of W250x45 steel beams against those of the CSA S16 and Eurocodes 3 design standards, it is observed that (i) If the effects of initial imperfections and residual stresses are excluded, the inelastic resistances are close to a fully plasticized section moment. In contrast, if the effects are included, the inelastic resistances are significantly smaller than the fully plasticized moment. (ii) The effects of initial imperfections on the moment resistance are significant for intermediate and long spans. Although the initial imperfection taken in the present study is 4.0 mm, that is within the allowable limit specified in the design standards (i.e., not greater than L/1000), the moment resistances with the taken imperfection are considerably smaller than the design moments specified in the design standards, and (iii) When considering steel beams with the effects of initial imperfection and residual stresses, the moment resistances based on the CSA S16 and EC3-6.3.2.3 solutions are higher, while those based on EC3-6.3.2.2 solution are lower than the moment capacities of the beams with the initial imperfection. This indicates that EC3-6.3.2.2 clause is the most safety design for the moment resistances.

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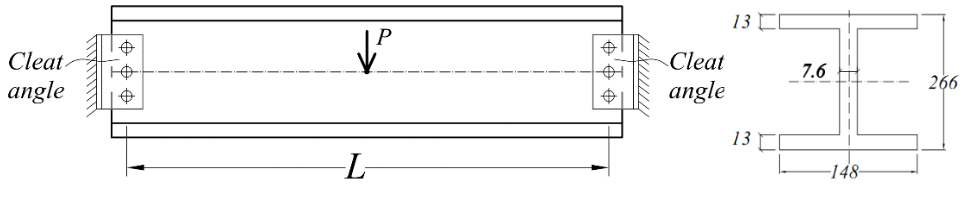
1 Introduction

Wide flange steel beams are widely applied to civil structures such as bridges, buildings and port structures, because they possess high shear and flexural strengths [1, 2]. Due to the wide application of such steel structures in civil structures, national design standards are published as standardized design guides [3, 4]. The failure modes of wide flange steel beams are relatively complicated and they depend on unbraced beam span length, web and flange class/compactness, and material. For steel beams with a class 1 or 2 section (or a compact section) and laterally unsupported, their failure mode may be based on a fully plasticized section when the unbraced length is short, an inelastic buck-ling resistance when the unbraced length is intermediate and an elastic buckling moment resistance when the unbraced length is long [3-5]. Among the above failure modes, the inelastic buckling resistance is complicated because they depend on residual stresses and imperfection initially stored in the beam [3-7]. Besides, both local and global buck-ling phenomena also depend on other design parameters such as steel Young modulus, Poisson's ratio, ratio of the width-thickness, and slenderness ratio. There were several numerical studies conducted to numerically evaluate the inelastic moment resistances of steel structures with taking the effects of residual stresses and initial imperfections [5-7]. Vales and Stan [5] focused on stochastic analyses of steel beam subjected to uniform bending. Residual stresses in their studies are incorporated into a numerical model through temperature deformations and stresses. Such a treatment might create initial stresses in the steel but they may not be identical to the designed residual stress model as given. It might also create initial deformation/strains in the steel those were not expected because the original steel beams were undeformed. Abebe et al. [6] focused on the inelastic buckling strength of steel columns while Elaiwi et al. [7] developed numerical solutions for castellated beams with holes on the web. In the context, the present study is going to conduct a numerical study based on ABAQUS [8] to investigate the effects of residual stresses and initial imperfections on the inelastic moment resistances of steel beams with classes 1 and 2. Both CSA S16 and Eurocodes 3 [3, 4] provides an allowable limit of the initial imperfection of L/1000 where L is the unbraced length of the beam. The present numerical study is going to investigate the moment resistances with such imperfection limits [3, 4].

2 Statement of the problem

A simply supported beam subjected to a point load P applied at the midspan section and at the sectional mid-height is considered (Fig. 1). The beam is laterally unsupported and it has a span of L and a prismatic W250x45 cross-section. Steel is assumed as a perfectly plastic material with an elastic modulus of E=200GPa, a yielding strength of F_y=350 MPa and a Poisson's ratio of 0.3. The effects of residual stresses and initial imperfections are considered. The present study is going to develop numerical models in ABAQUS those capture the residual and imperfection effects. Then, elastic and inelastic moment resistances based on different spans predicted by the present numerical study are compared to those of the Canadian (CSA-S16) [3] and Eurocodes 3 [4] standards.



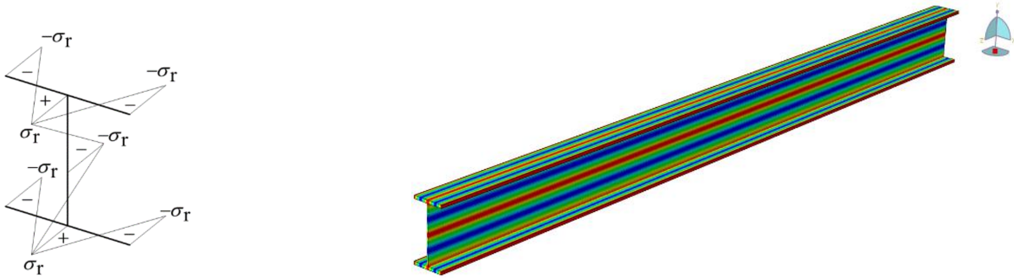
a) Beam profile b) W450x45 section

Fig. 1 – A simply supported beam subject to a midspan point load

3 Modelling of the structure in ABAQUS

Although the modelling of such a steel beam in ABAQUS is relatively simple, how-ever the incorporating of the effects of residual stresses and initial imperfections into the model may be a challenge for engineers. The present part aims at showing a technique to incorporate the nonlinear effects into the ABAQUS models. To accurately capture the responses of the model in elastic buckling problems and the inelastic moment resistance problems, ABAQUS models developed in the present study are based on brick element C3D8R in the ABAQUS library [8]. The element has 8 nodes with three translations per node, totaling 24 DOFs and adopts reduced integration to avoid volumetric locking, and thus has a single integration point located at the element centroid.

Implementation of residual stresses: Figure 2 presents a model of residual stresses distributed on a beam cross-section, in which the value of σ_r is taken as $0.3\ F_y=105\ MPa$ as indicated in design standards [3,4]. They exist in the shaped-steel products and they may not be neglected in designing. The residual stresses are incorporated into the ABAQUS models through the *INITIAL CONDITIONS, TYPE=STRESS keyword. They are assumed constant throughout the section thickness. A blank *STEP is then set to balance stresses in the steel, before the loading step is evoked.



a) Residual stresses model b) Residual stresses in ABAQUS models in the present study

Fig. 2 – Residual stress implemented in the ABAQUS models in the present study

Implementation of initial imperfections: In Canadian code (CSA S16), an initial imperfection of the beam axis of L/1000 is allowed. To incorporate the initial imperfection into the ABAQUS models, there are several methods such as using the first buckling mode or directly changing node coordinates. The present study implements the initial imperfections by following the method of the first lateral-torsional buckling mode. The initial imperfection is based on the first mode with a magnitude factor of 4 so as to introduce a peak imperfection of 4 mm at the midspan (Fig. 3). The two command lines are required as *IMPERFECTION, FILE=A4mBareElasticbuckling, STEP=1 and 1,4.

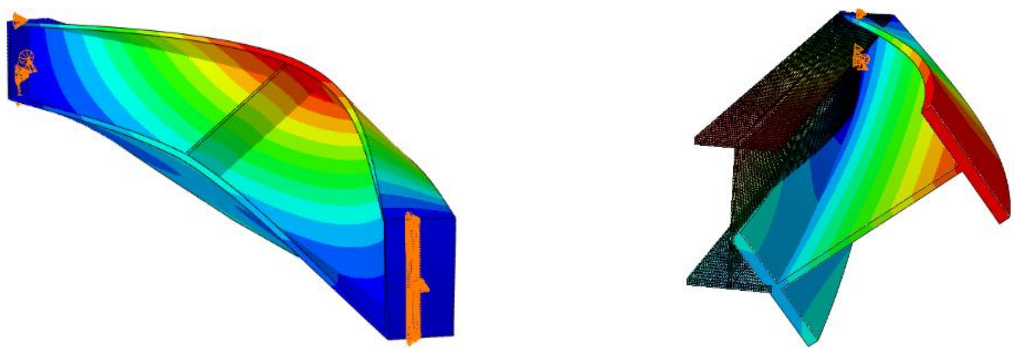


Fig. 3 – Implementation of initial imperfection through the first lateral-torsional buckling mode

The FEA analyses are conducted in ABAQUS to provides (1) elastic buckling moment resistances M_u and (2) inelastic moment resistances M_r of the steel beams with/without considering the effects of the residual stresses and the initial imperfections.

The elastic buckling analyses are based on keyword *Buckle in *STEP level. It is noted that in the analysis of elastic buckling problems, three web stiffeners at the two beams ends and at the midspan are added to avoid/ reduce web distortion effects [2].

To obtain inelastic moment resistances, two different analyses are conducted and denoted as "FEA1-WithMat-NoIM-NoR", "FEA2-WithMat-WithIM4mm-WithR". In the "FEA1-WithMat-NoIM-NoR" analysis, material nonlinearity is included but initial imperfection and residual stresses are excluded. In the "FEA2-WithMat-WithIM4mm-WithR" analysis, material nonlinearity, initial imperfection and residual stresses are included, in which the magnitude of the peak imperfection at midspan is 4.0 mm. Both FEA analyses are based on RIKs method through keyword *STATIC, RIKS in combining with nonlinear geometric effects through *STEP, NLGEOM=YES. The number of increments, times step, the maximum and minimum iteration bounds are set as 30, 0.005, 1.0, 1e-008 respectively. It is also noted that in the analyses of inelastic moment resistances, web stiffeners are excluded in the FEA model so that the analyses can capture local web buckling modes. In the FEA solutions, the moment resistance M_r is equal to the lower value of the elastic moment resistance M_u and the inelastic moment resistance M_{in} those are determined as discussed above.

4 Factored moment resistances based on CSA-S16 specification [3]

The factored moment resistance, M_r, of the beam shall be determined as follows: When $M_u > 0.67M_p$: $M_r = 1.15\phi M_p\left[1-0.28M_p/M_u\right] \le \phi M_p$ and when $M_u \le 0.67M_p$: $M_r = \phi M_u$. Here the elastic buckling moment resistance is evaluated as $M_u = (\omega_2\pi/L)\sqrt{EI_yGJ+(\pi E/L)^2 I_yC_w}$ while the fully plasticized moment $M_p = ZF_y$.

5 Factored moment resistances based on Eurocode 3 specification [4]

For a double symmetric cross-section with classes 1 and 2 sections and the beam is laterally unsupported, the factored moment resistance, M_r , of the beam shall be deter-mined as follows $M_r = \chi_{LT}(ZF_y/\gamma_{MI})$ in which γ_{MI} is safety factor and it is set as 1.0 in the present study. χ_{LT} is the reduction factor for lateral-torsional buckling and it should not be greater than 1.0. Eurocodes 3 provides two solutions for χ_{LT} based on Clauses 6.3.2.2 and 6.3.2.3 [4].

6 Result discussions

Based on the present developed finite element model (denoted as FEA solutions), the elastic buckling moments and inelastic moment resistances of the steel beam with different span lengths $L = 2.0, 4.0, 6.0\ m$ are evaluated. The results are then compared against the design moments by code equations based on CSA S16 [3] and Eurocodes 3 [4]. Based on CSA S16, the moment resistance of the beam depends on span lengths. For the steel section taken, the fully plastic section moment $M_p = 211 kN.m$ governs the beam failure when the beam span $L \le 2.58\ m$. The inelastic buckling moment M_r governs the beam failure when the beam span $2.58\ m \le L \le 5.90\ m$. And the elastic buckling moment M_u governs the beam failure when the beam span $L \ge 5.90\ m$. The plastic moment M_p and inelastic buckling moment M_r respectively account for the resistance of the beam based on material failure and local buckling modes.

6.1 Elastic buckling moment resistance Mu and FEA model verification

Figure 4 presents the elastic buckling moment resistance M_u against the span length ranged from 2.0 to 7.0, as obtained from the present FEA solution and from the CSA S16 code [3].

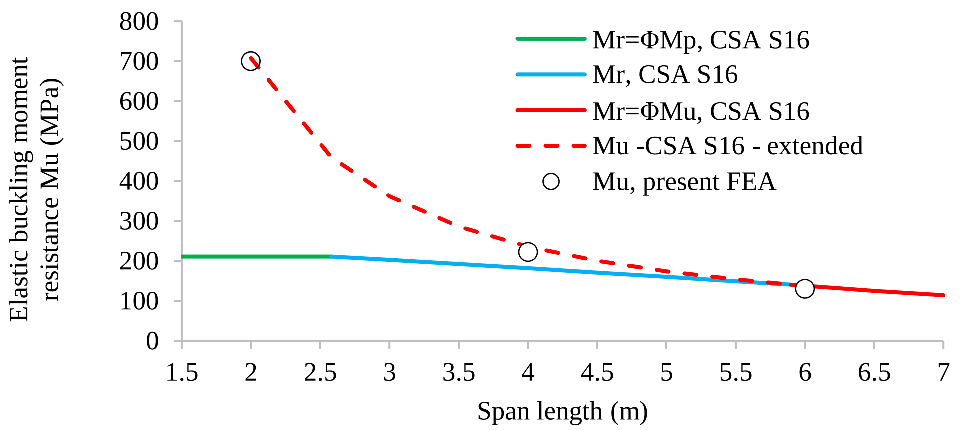


Fig. 4 – Comparison of the elastic buckling moment resistance between the present study against CSA-S16 specification

Overlaid on the figure is the extended elastic moments (denoted as “Mu-CSA S16-extended”) based on CSA S16 code for beams with span shorter than 5.90 m. The lateral torsional buckling configuration is similar to those provided in Fig. 3. Although such elastic buckling moments do not govern the system failure because they are greater than inelastic moments M_p, M_r, they are here provided to verify the elastic buckling moments predicted by the present developed FEA solution as well as they are taken as an initial imperfection shape for the subsequent inelastic buckling FEA analyses. Regardless to the limits created by M_p and M_r, the elastic buckling moments as obtained from the present FEA solution are found to excellently agree with those pro-vided by the CSA S16 solution for spans $L = 2.0, 4.0, 6.0\ m$.

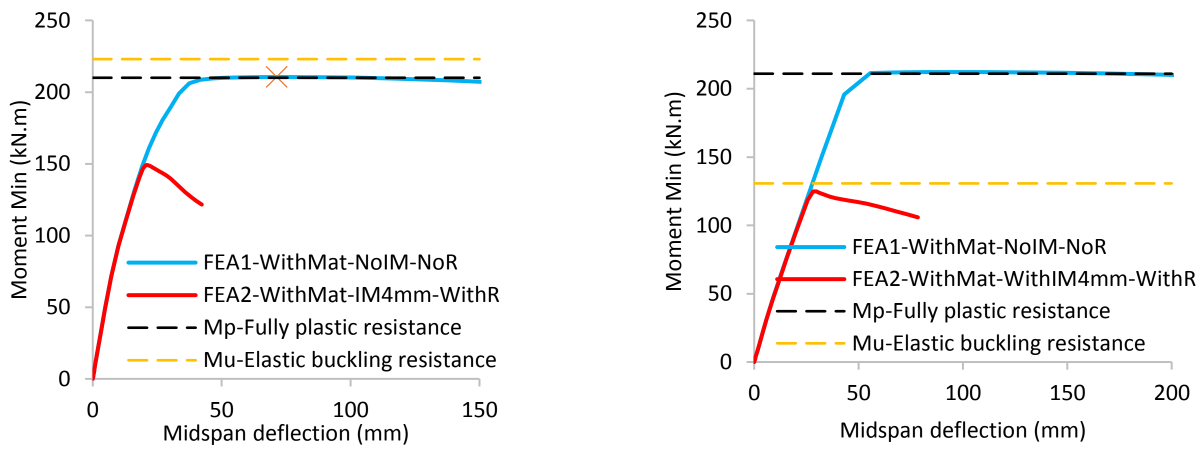
Table 1 presents the moment values and the differences between the two solutions. The differences between the two solutions are within 7.1%. As also observed in Fig. 4, the elastic buckling moments M_u for spans $L = 2.0$ and $L = 4\ m$ are higher than the inelastic moments M_r and M_p and thus moments M_u do not govern the beam failure. In contrast, the elastic buckling moment M_u for span $L = 6\ m$ is less than the inelastic moments and thus it governs the system failure.

Table 1 - Comparisons of M_u between the CSA S16 and present FEA solutions

L (m)	Mu-CSA S16	Mu-FEA	% difference
2.0	707.7	700	1.1
4.0	235.8	223	5.4
6.0	140.7	130.7	7.1

6.2 Inelastic moment resistance M_{in}

Figures 5a, b present the inelastic moment resistance M_{in} for spans $L = 4.0, 6.0\ m$ against midspan deflection, as predicted by the present “FEA1-WithMat-NoIM-NoR”, “FEA2-WithMat-WithIM4mm-WithR” solutions. Also, overlaid on the figures are the fully plastic moment resistance M_p and elastic buckling resistances M_u as evaluated in Sections 4 and 5. For span $L = 4.0\ m$, one has $M_p = 211 kN.m$ and $M_u = 235.8\ kN.m$.



a) Span L=4.0m

b) Span L=6.0m

Fig. 5 – Inelastic moment- midspan deflection relationships based on different inelastic FEA analyses (a) Span L=4.0m and (b) span L=6.0m

The FEA1 solution provides a nearly constant moment resistance that is close to the fully plastic section moment M_p while the peak moment of the FEA2 solution is significantly smaller than the moments M_p and M_u. The inelastic moment resistance based on the FEA2 solution is 149.2 kNm. For span $L = 6.0m$, one has $M_p = 211 kN.m$ and $M_u = 140.7\ kN.m$. Again, the FEA1 solution provides a constant moment resistance that is close to moment M_p. Meanwhile, the peak moment of the FEA2 solution is found to be considerably smaller than moments M_p and M_u. The inelastic moment resistance based on the FEA2 solution is 124.9 *kNm*. Through the above discussions, it is observed that (i) If the effects of initial imperfections and residual stresses are excluded, the inelastic moment resistances in the FEA solutions are close to the fully plastic section moment M_p. In contrast, if the effects are included, the inelastic moment resistances are significantly smaller than the plastic moment, and (ii) The moment resistance of the FEA2 solution is smaller than the elastic buckling moment resistances M_u predicted by the CSA S16 [3] and the present FEA solutions (i.e., $M_{u,CSA} = 140.7 MPa$ and $M_{u,FEA} = 130.7 MPa$ as summarized in Table 1). Thus, the failure mode of the FEA2 solution is governed by inelastic moment resistances. However, the CSA S16 solution [3] indicates that the failure mode of the beam is governed by the elastic buckling resistance.

6.3 Moment resistance M_r

Figure 6 presents, and Table 2 summarizes, the moment resistances M_r against different span lengths, as designed in CSA S16 code [3], Eurocodes 3 [4] and predicted by the present FEA solutions. As discussed, the moment resistance based on CSA S16 [3] is $M_r = \phi M_p$ when the beam span $L \le 2.58\, m$, it is $M_r = 1.15\phi M_p \left[1 - 0.28 M_p / M_u\right] \le \phi M_p$ when the beam span $2.58\, m \le L \le 5.90\, m$ and it is $M_r = \phi M_u$ where M_u is the elastic buckling moment when the beam span $L \ge 5.90\, m$. The moment resistance M_r based on Eurocodes 3 [4] based on Clauses 6.3.2.2 and 6.3.3.3. In the present FEA1 and FEA2 solutions, the moment resistance M_r of the system is based on the lower value of the elastic moment resistance M_u and the inelastic moment resistance M_{in} (Table 2).

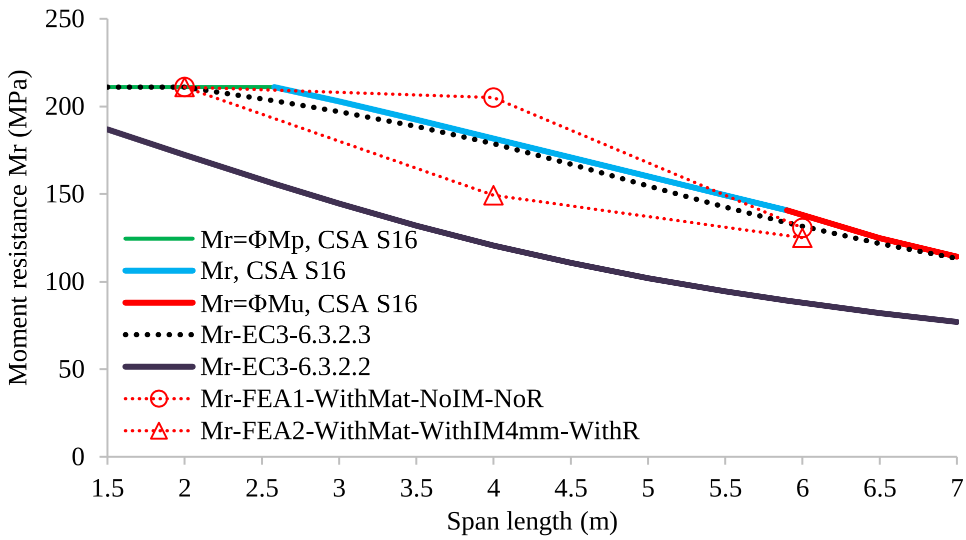


Fig. 6 – Comparison of the inelastic moment resistances between the present study against CSA-S16 and Eurocodes standards [3,4]

Table 2 - Comparison of moment resistance against different spans between different solutions

L_u (m)	M_u-FEA	M_{in}-FEA		M_u-FEA = min(M_u-FEA, M_{in}-FEA)		M_r based on codes		
		FEA1	FEA2	FEA1	FEA2	CSA S16	EC3-6.3.2.2	EC3-6.3.2.3
2.0	700	211	211	211	211	211.0	172.4	211.0
4.0	223	210.5	149.2	210.5	149.2	181.7	120.7	178.7
6.0	130.7	210.5	124.9	130.7	124.9	140.7	89.1	133.6

Among all solutions, the CSA S16 solution is taken as a reference solution. For span $L = 2.0\ m$, the CSA S16 solution predicts a system failure based on a fully plastic section mode with $M_r = \phi M_p = 211\ kNm$. This is also the predictions of the present FEA1, FEA2, and EC3-6.3.2.3 solutions. For span $L = 4.0\ m$, the CSA S16 solution predicts an inelastic buckling mode with a moment resistance of $M_r = 181.7\ kNm$. The FEA1 and FEA2 solutions also predicts inelatic buckling modes. However, the moment resistance of the FEA1 solution is $210.5\ kNm$, which is higher than that of the CSA S16 solution. In contrast, the moment resistance based on FEA2 solution is 149.2, that is significantly smaller than the moment resistance of the CSA S16 solution. Similar observations in span $L = 4.0\ m$ are obtained for span $L = 6.0\ m$. The moment resistances of the EC3-6.3.2.3 are relatively similar to those of the CSA S16 solution. Based on the above observations, it is commented that (iii) Because the FEA2 solution includes the effect of initial imperfections and residual stresses while the FEA1 solution excludes the effects, the effects of initial imperfections and residual stresses on the moment resistance of the beam are thus significant for intermediate and long spans (e.g., $L = 4.0\ m$ and $L = 6.0\ m$), (iv) Although the initial imperfection taken for the beam is $4\ mm$ and it is within the allowable limit specified in the CSA S16 standard (i.e., not greater than $L/1000$), the moment resistances of the beam with the taken imperfection are considerably smaller than the moment resistances Mr specified in CSA S16 and EC3-6.3.2.3 codes, and (v) When considering steel beams with the effects of initial imperfection and residual stresses, the moment resistances based on the CSA S16 and EC3-6.3.2.3 solutions are higher, while those based on EC3-6.3.2.2 solution are smaller than the moment capacities of the FEA2 solution. This indicates that the EC3-6.3.2.2 clause is the most safety design for moment resistance of the given steel beams.

7 Conclusions

Elastic and inelastic moment resistances of wide flange steel beams without/with considering the effects of initial imperfections and residual stresses are numerically investigated in the present study. The numerical study is conducted in ABAQUS in which residual stresses are incorporated by using initial conditions keyword while initial imperfection is imported by using the first lateral-torsional buckling mode imperfection key-word. The present FEA models are then adopted to predict (1) elastic buckling moment resistances M_u and (2) inelastic moment resistances M_{in} of the steel beams with/without the effects of residual stresses and initial imperfections. By comparing the FEA moment resistances M_r of W250x45 steel beams against the those in CSA S16 and Eurocodes 3 codes [3, 4], key conclusions are summarized in the following.

(i) The elastic buckling moment resistance based on the present FEA solutions are in excellent agreements with those of the CSA S16 and Eurocodes 3 solutions [3,4].

(ii) If the effects of initial imperfections and residual stresses are excluded, the inelastic moment resistances of the beams are close to the fully plastic section moment M_p. In contrast, if the effects are included, the inelastic moment resistances are significantly smaller than the plastic moment and they govern the system failure. The characteristic of the inelastic failure is local web buckling.

(iii) The failure mode of the steel beam with span $L = 6.0m$ with taking the effects of initial imperfections and residual stresses is governed by inelastic moment resistances in the FEA solution. However, the CSA S16 [3] indicates that the failure mode of the beam is governed by an elastic buckling resistance.

(iv) The effects of initial imperfections and residual stresses on the moment resistance of the beam are significant for intermediate and long spans (e.g., $L = 4.0$ and $L = 6.0$). Although the initial imperfection taken in the present study is $4.0\ mm$ that is within the allowable limit specified in CSA S16 code (i.e., not greater than $L/1000$), the moment resistances of the beam with the taken imperfection are considerably smaller than the design moment resistances in CSA S16 and EC3-6.3.2.3 standards [3, 4], and M_{in}

(v) When considering steel beams with the effects of initial imperfection and residual stresses, the moment resistances based on EC3-6.3.2.2 solution [4] are the smallest ones. This indicates that the EC3-6.3.2.2 code [4] I the most safety design for the moment resistance of the given steel beams.

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