CYCLIC OUT-OF-PLANE INSTABILITY OF DEEP WIDE-FLANGE STEEL BEAM-COLUMNS

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1. ABSTRACT

The current steel construction practice in North America concentrates the lateral load resisting system in the perimeter of a plan view. For the case of steel moment resisting frames (MRFs) this leads to selection of deep and slender wide-flange members in order to satisfy the lateral drift requirements by the design code and to respect economy. These members have low out-of-plane moment of inertia and high web and flange slenderness ratios. Such sections are vulnerable to severe local and out-of-plane global instabilities when subjected to large lateral drifts and high axial load levels. To this end, a detailed finite element study is employed to investigate the cyclic behaviour of a wide range of deep wide-flange sections, as part of typical first-story interior MRF columns. Emphasis is placed on the effect of the web and flange slenderness on: 1) the steel column stiffness deterioration, 2) out-of-plane displacements, and 3) the lateral brace force demands. The results show that deep slender beam-columns can lose 80% of their flexural stiffness at 4% chord-rotation. Deep beam-columns also experience out-of-plane displacements of about 6% of their length. Finally, it is shown that low out-of-plane force demands are observed at the floor-level since the column is free to twist and buckle within the story height.

2. INTRODUCTION

In the current construction practice of steel buildings with perimeter special moment resisting frames (SMFs) designed in the West Coast of North America, columns are typically assigned deep (depth larger than 400mm) wide-flange steel sections. These sections have high in-plane moment-of-inertia to weight ratio. Therefore they represent an economic choice while satisfying the design code requirements such as the strong-column-weak-beam (SCWB) criterion. However, these sections have low out-of-plane moment-of-inertia and web and flanges with high slenderness ratios near to the compactness limit specified by [1] for highly ductile members. This makes them vulnerable to local buckling, twisting, and out-of-plane instabilities. In SMFs, out-of-plane lateral bracing is only provided to the columns at the floor levels. Within a given story, the columns are free to twist and buckle out-of-plane. In the case of a seismic event, cyclic deterioration in the column's stiffness will take place due to lateral torsional buckling. Experimental research on fully-restrained beam-to-column connections [2, 3] showed that when deep members are used as columns in Reduced Beam Section (RBS) connections (typically used in SMFs), column twisting is observed due to inelastic beam instability.

In this paper, a detailed finite element (FE) model of a typical first-story interior SMF column is used to investigate the cyclic behaviour of 35 deep wide-flange steel sections when subjected to symmetric cyclic lateral loading combined with different levels of compressive axial load ratios. The effect of section slenderness on the cyclic deterioration in stiffness, out-of-plane displacements, and lateral brace force demands is investigated.

3. PROPOSED FINITE ELEMENT MODEL FOR STEEL BEAM-COLUMNS

A detailed finite element model is developed in the FE analysis software ABAQUS/CAE [4] for a typical first-story interior steel SMF column. Figure 1(a) shows the FE model and the applied boundary conditions. The column has a length (L) of 4.6m (15 feet), fixed boundary conditions at the base and flexible boundary conditions at the top end. The flexible boundary condition reflects the actual boundary of a first-story SMF column where there is rotational flexibility provided by the beam-to-column connection at this location. These boundary conditions can capture the changes in the moment gradient due to the movement of the inflection point after the occurrence of column yielding and/or local buckling. The flexible boundary conditions are simulated using an elastic beam element connected to the column's top end. The elastic beam element is assigned a pre-calculated in-plane moment-of-inertia that enforces the inflection point to be at 0.75 L, measured from the base, during elastic loading (see Figure 1(a)). This assumed location of the inflection point is expected in first-story interior SMF columns when they remain elastic [5].

Typical S4R shell elements are assigned to the FE model to capture the web and flange local buckling. The shell elements are assigned a nonlinear isotropic/kinematic hardening material model. The parameters of this model are defined based on the measured engineering stress-strain curve obtained from uniaxial coupon testing. Therefore, these parameters are independent of the loading history experienced by the column. In this study, the engineering stress-strain and constant amplitude cyclic curves conducted by [6, 7] are used to define the input parameters of the steel material model. The FE model is validated with past experimental studies [8, 9]. Figures 1(b) and (c) show a sample comparison between the moment-rotation relation predicted by the FE simulation and the experimental data for specimen C5 by [9] and specimen W14x176-35 by [8], respectively. The FE model can adequately capture the cyclic deterioration in strength and stiffness of a steel beam-column due to complex section local instabilities. For additional details regarding the FE modeling approach and the validation of the FE model, the reader should refer to [10].

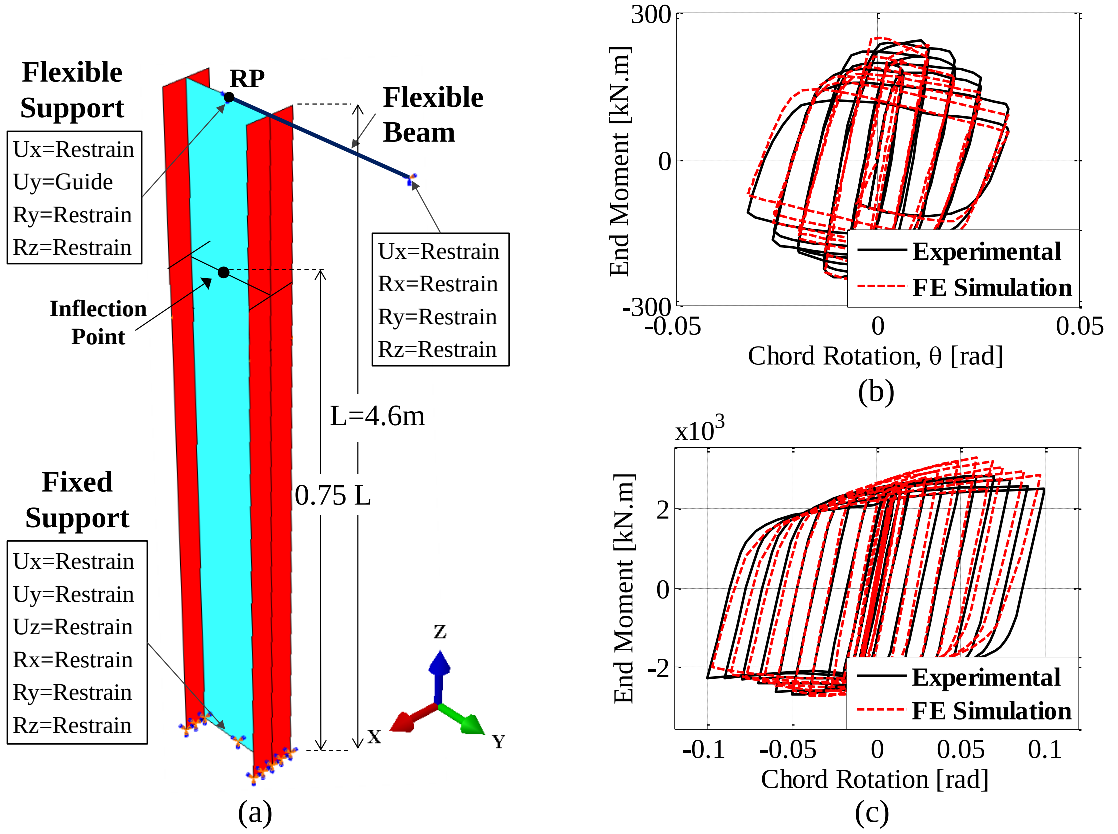


Fig. 1 (a) Boundary conditions applied to the FE model; (b) comparison between the experimental end moment-chord rotation relation and the FE predicted response: specimen C5 [data by [9]]; (c) specimen W14x176-35 [data by [8]]

A set of 35 wider-flange sections is employed in the present study. The 35 sections are compact as per AISC-341-10 [1] compactness limits for highly ductile members (λ_{hd}). These sections cover different wide-flange sizes including those commonly used in the current construction practice of SMFs in North America. The detailed list of the sections can be found in [10] but not included here due to brevity. The column is subjected to the SAC symmetric cyclic lateral loading protocol [11]; applied in displacement control at the top end of the columns. The lateral loading is combined with different levels of constant compressive axial loads ranging from 0% to 50% P_y, where P_y is the axial yield strength of the corresponding column. Note that during a seismic event, first-story interior SMF columns experience lesser variation in their applied axial loads due to overturning moments compared to end columns. This justifies the use of constant compressive axial loads in the present analytical study.

4. CYCLIC DETERIORATION IN BEAM-COLUMN FLEXURAL STIFFNESS

Figures 2(a) and (b) show the end moment versus the chord rotation for the W30x391 and W30x148 columns, respectively, when subjected to an axial load equal to 20% P_y. The W30x391 column has low web and flange slenderness (h/t_w=19.7 and $b_f/2t_f$=3.19) while the W30x148 column has high web and flange slenderness (h/t_w=41.6 and $b_f/2t_f$=4.44).

However, the latter is still a highly ductile member according to AISC-341-10 seismic provisions. Figure 2 shows that at a chord-rotation of 4% radians, the W30x391 column's flexural stiffness is reduced by 35% compared to a 75% reduction for the W30x148 column. This is attributed to the low torsional constant, J, and weak axis radius of gyration, r_y, of the W30x148 column compared to those of the W30x391 column.

Figures 3(a) and 3(b) show the progression of the out-of-plane displacement at different chord-rotations for the W30x391 and W30x148 columns, respectively. Figure 3 demonstrates the severity of the out-of-plane displacements in the more slender column (i.e., W30x148). Out-of-plane displacements equal to 6% L at 4% radians are observed for the W30x148 column compared to about 1% L for the W30x391 column. The out-of-plane displacements, in addition to local buckling, accelerates the cyclic deterioration in strength as observed in Figure 2 as well as the column axial shortening as shown in Figure 3(b).

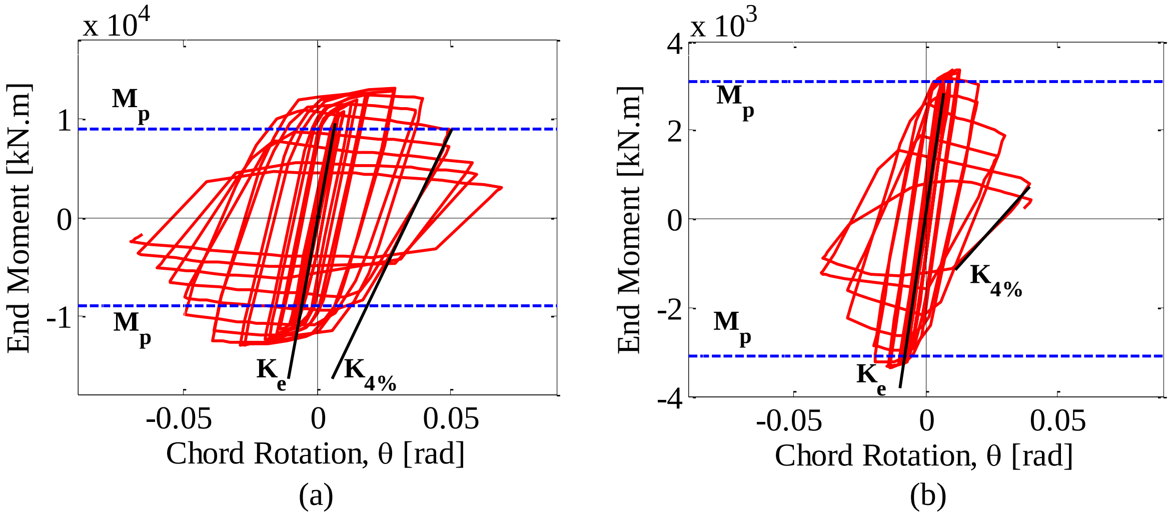
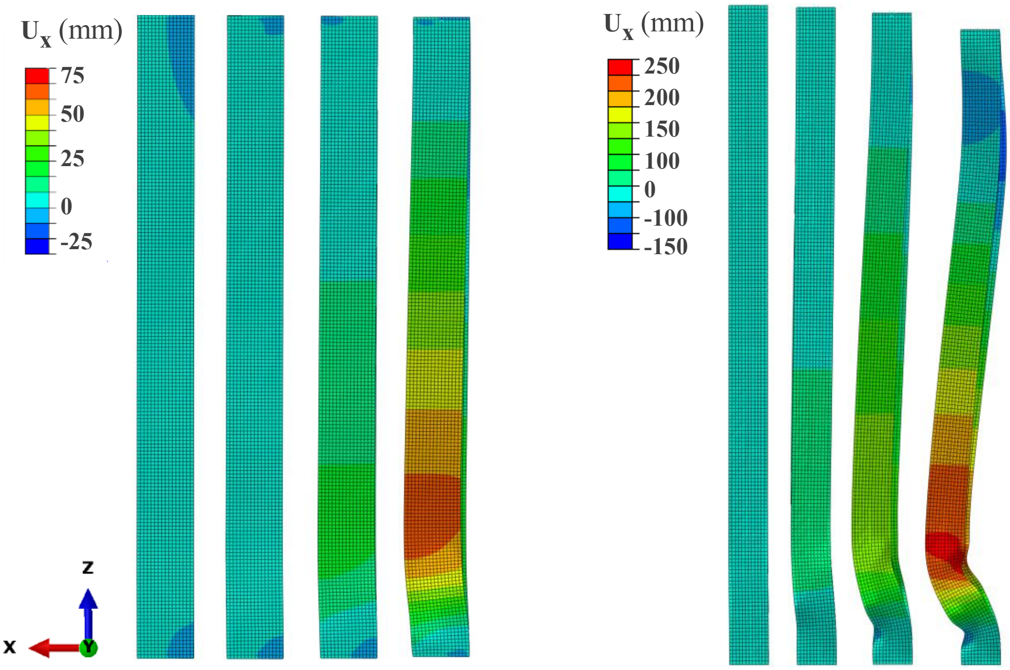


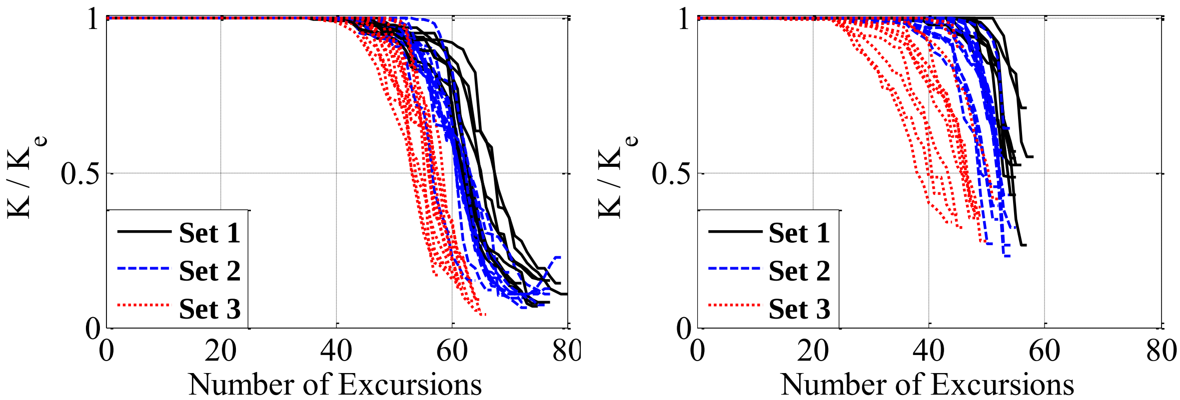
Fig. 2 End moment at column base versus chord rotation when subjected to 20% P_y: (a) W30x391; (b) W30x148 steel beam-columns

The deterioration in flexural stiffness occurs at lower chord-rotations when the beam-column is subjected to higher axial load ratios. In first-story interior columns, axial load ratios of up to 50% P_y are expected in high-rise steel buildings [12, 13]. Figures 4(a) and (b) show the flexural stiffness K of each beam-column, normalized by the corresponding initial elastic flexural stiffness K_e, versus the number of loading excursions for the 35 beam-columns when subjected to 20% and 50% P_y, respectively. It should be noted that for the SAC protocol, excursion number 60 corresponds to the second cycle of 4% lateral drift ratio. In Figure 4, the sections are divided into 3 sets based on the web and flange slenderness, where Sets 1, 2, and 3 represent compact sections (λ_{hd}) with low, moderate and high slenderness ratios, respectively. Figure 4 shows that the flexural stiffness of all the beam-columns under consideration is sustained for several excursions before decreasing due to the occurrence of local and lateral torsional buckling. The same figure also shows that at a given axial load ratio and excursion number, the more slender sections experience severe stiffness deterioration. For example, at 20% P_y and 60 excursions, Set 1 beam-columns (h/t_w <22 and $b_f/2t_f$ <3.9) experience an average loss in their flexural stiffness of 20% compared to 80% for Set 3 (32.5< h/t_w <43 and 5.5< $b_f/2t_f$ <7.0). Furthermore, the rate of flexural stiffness reduction is higher at higher load ratios. This is indicated by the steeper curves in Figure 4(b) compared to those in Figure 4(a). This is attributed to the more severe out-of-plane displacements that are amplified by the higher P-Δ effects about the column's weak-axis, where Δ is the out-of-plane displacement. Note that at 50% P_y (see Figure 4(b)), all beam-columns lost their flexural strength after undergoing less than 60 excursions (i.e., less than a chord rotation of 4% radians).



(a) (b)

Fig. 3 Progress of out-of-plane displacement at 1%, 2%, 3%, and 4% chord rotation when subjected to 20% P_y: (a) W30x391; (b) W30x148



(a) (b)

Fig. 4 Normalized flexural stiffness versus number of loading excursions: (a) 20% P_y; (b) 50% P_y

5 OUT-OF-PLANE FORCE DEMANDS

The reaction force at the lateral support at the column top end (P_{brace}) is monitored in order to evaluate the out-of-plane force demands resulting from column twisting due to lateral torsional buckling. These forces are exerted to act axially at the corresponding gravity beam that acts as a lateral brace for the column in its orthogonal direction. Figures 5(a) and (b) show the values of the P_{brace}, normalized by P_y, versus the L/r_y ratio when the columns are subjected to 20% and 50% P_y, respectively. In the figure, the limit for member slenderness specified by AISC-341-10 [1] is superimposed (i.e., $L/r_y < 60$). Note that the

P_{brace} values are the maximum forces measured up to a reference chord rotation of 4% radians and that L is the same for all 35 beam-columns (i.e., L=4.6m) since lateral bracing is only provided at the floor level. Figure 5 shows that there is practically no correlation between the lateral brace force demand and the L/r_y ratio. Beam-columns with lower r_y, and similar torsional constants J, experience lower brace force demands. This is attributed to the weak resistance of the beam-columns with low r_y and J to out-of-plane deformations and twisting within the column height.

For the 35 beam-columns considered in the present study, an average brace force demand of 0.3% P_y was measured at 20% P_y (see Figure 5a). At 50% P_y, the lateral brace force demand reached up to about 0.9% P_y for sets 1 and 2 (see Figure 5(b)). Furthermore, the values of the lateral brace force demand is about 50% lower than the required brace axial strength specified for beam-columns as per AISC/ANSI 360-10 [14]. In a recent study by the authors [15], it was shown that if lateral bracing is provided within the story height as per the [14] unbraced length requirements, lateral brace force demands exceed those specified by [14].

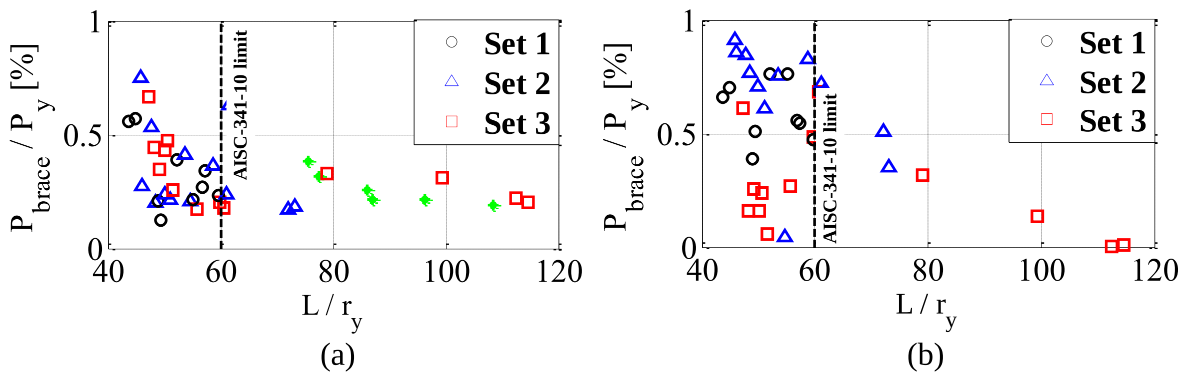


Fig. 5 Normalized lateral brace axial force, measured at 4% chord rotation, versus L/r_y ratio: (a) 20% Py;(b) 50% Py

5. SUMMARY AND CONCLUSIONS

This paper investigates the cyclic out-of-plane instability of first-story interior SMF columns that are assigned deep wide-flange sections. A detailed FE model of a steel beam-column is developed and validated in order to investigate the cyclic behaviour of a set of 35 deep wide-flange beam-columns. The set of beam-columns is subjected to symmetric cyclic lateral loading combined with different levels of constant compressive axial load ratios. The following observations are made:

- At P/P_y = 20%, the flexural stiffness of slender wide-flange beam-columns with 32.5<h/t_w<43 and 5.5<$b_f/2t_f$<7 is reduced by 80% at chord rotation of 4% radians compared to 20% for highly compact wide-flange beam-columns with h/t_w<22 and $b_f/2t_f$<3.9.
- The rate of cyclic deterioration in flexural stiffness is larger at higher axial load levels due to the severe out-of-plane displacements amplified by higher axial loads.
- Out-of-plane displacements of about 6% L are observed in slender wide-flange beam-columns at 4% radians compared to 1% L for highly compact sections.
- The lateral bracing force demands exerted at the floor level (i.e., column's top end) do not correlate with the beam-column section slenderness. However, these forces do correlate with the out-of-plane radius of gyration r_y and the torsional constant J,

where beam-column with lower r_y and J experience lower brace force demands. Relatively low out-of-plane forces with a maximum of 1% P_y are observed.

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ΠΕΡΙΛΗΨΗ

Η υπάρχουσες κατασκευαστικές διατάξεις μεταλλικών κτιρίων στη βόρεια Αμερική θεωρούν ως βέλτιστη οικονομική λύση την τοποθέτηση πλαισίων όπου παραλαμβάνουν τις σεισμικές δράσεις μόνο στην περίμετρο ενός κτιρίου. Στην περίπτωση πλαισίων κάμψης, η πρακτική αυτή οδηγεί στην επιλογή υψίκορμων διατομών για τον σχεδιασμό των υποστυλωμάτων έτσι ώστε να ικανοποιούνται οι απαιτούμενοι έλεγχοι των πλευρικών μετατοπίσεων από άνεμο και σεισμό. Οι διατομές αυτές παρόλο που κατατάσσονται ως διατομές κατηγορίας 1, έχουν συνήθως μικρή ροπή αδράνειας εκτός επιπέδου φόρτισης και σχετικά υψηλές τιμές τοπικών λυγιροτήτων κορμού και πέλματος. Αυτό έχει ως αποτέλεσμα οι διατομές αυτές να είναι ευάλωτες σε φαινόμενα πλαστικού τοπικού και στρεπτοκαμπτικού λυγισμού όταν υποβάλλονται σε πλευρικές μετατοπίσεις λόγω σεισμού σε συνδυασμό με υψηλά αξονικά φορτία. Η παρούσα εργασία μελετά τη συμπεριφορά μεταλλικών υποστυλωμάτων υπό ανακυκλιζόμενη φόρτιση με χρήση αναλυτικών προσομοιωμάτων με πεπερασμένα στοιχεία. Με βάση τα αποτελέσματα, υψίκορμες διατομές με όρια τοπικής λυγιρότητας κοντά στα υπάρχοντα επιτρεπτά όρια του αμερικάνικου κανονισμού χάνουν περισσότερο από το 80% της πλαστικής αντοχής τους σε κάμψη όταν οι πλευρικές μετακινήσεις του αντίστοιχου ορόφου στον οποίο ανήκουν ξεπεράσουν το 4%.