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EXPERIMENTAL INVESTIGATION OF THE HYSTERETIC BEHAVIOR OF WIDE-FLANGE STEEL COLUMNS UNDER HIGH AXIAL LOAD AND LATERAL DRIFT DEMANDS

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Abstract. *This paper discusses the findings from a large-scale experimental program that characterized the hysteretic behavior of typical steel wide-flange columns in steel moment-resisting frames (MRFs). The test specimens were tested in a cantilever configuration with a fixed point of inflection. The main testing parameters included various lateral and axial loading histories, the applied axial compressive load and the local slenderness of the cross-section. It is shown that (a) steel columns subjected to high compressive axial loads (i.e., larger than 50% of their critical axial load) should not be treated as forced-controlled elements as suggested by ASCE/SEI 41-13;(b) the axial shortening is an important deterioration mode that should be explicitly considered as part of the seismic design process of columns in steel MRFs; (c) end columns are characterized by non-symmetric hysteretic behavior due to the dynamic overturning effects during an earthquake. The test program provided unique experimental data that characterized the monotonic backbone curve of steel columns through the loss of their axial load carrying capacity under various levels of axial load ratios.*

1 INTRODUCTION

Current nonlinear seismic assessment procedures in North America treat columns in new and existing steel frame buildings as force-controlled elements (i.e., zero plastic deformation capacity) if the column compressive axial load demands become larger than 50% of the critical load of the respective member [1]. Bech et al. [2] has shown that this assumption can become critical in cases that effective retrofit solutions are examined for existing steel frame buildings when their columns experience high axial compressive loads during an earthquake. Recent tests on wide-flange steel columns examined the effect of local slenderness, loading protocols and boundary conditions on the hysteretic behavior of wide-flange steel columns under moderate compressive axial load ratios [3–5]. Limited experimental evidence primarily from small scale wide-shape steel columns suggests that these members may have an appreciable plastic deformation capacity even in cases that they are subjected to high axial load demands [6]. Therefore, there is a need to characterize experimentally the hysteretic behavior of steel columns subjected to high axial loads and lateral drift demands. More recently, the earthquake-induced collapse risk quantification of frame buildings has gained increased attention [7–9]. In this context, a number of researchers [10–13] have highlighted the lack of monotonic tests that push structural components far into the inelastic range in order to properly quantify their ultimate deformation capacity. The monotonic backbone curve can be treated as a characteristic property of a structural component unlike the first-cycle envelope curve that is loading history dependent.

This paper summarizes the findings from a large-scale experimental program that investigated the hysteretic behavior of wide-flange steel columns that are commonly used in steel moment-resisting frame

(MRFs) designed in seismic regions. Emphasis was placed on the effects of high axial compressive loads on both the monotonic and cyclic response of steel columns. The effect of the employed loading history on the hysteretic behavior of steel columns was also assessed including cases that the applied axial load varied in synchronization with the lateral drift demands. This is a typical loading condition seen in end columns as part of MRFs due to dynamic overturning effects.

2 TEST MATRIX AND EXPERIMENTAL SETUP

The test matrix is summarized in Table 1. It consists of three sets of cross-section sizes including a W14x61, W14x82 and a W16x89. Each set includes four nominally identical steel columns fabricated by ASTM A992 Grade 50 steel (i.e., nominal yield stress, f_y = 345MPa). The test specimens are selected by considering (a) the local slenderness ratios of highly compact cross-sections as per AISC 341-10 [14]; and (b) commonly used cross-sections in typical mid-rise steel frame buildings with MRFs [15,16] at a two-third scale given the equipment limitations at McGill University testing facilities. The steel columns are tested in a cantilever configuration based on the assumption that the inflection point in the column remains constant. The length of each specimen is 1875mm.

Table 1. Test Matrix.

Specimen ID	Cross-Section	Lateral loading	Axial loading protocol
W-8-34-M-C-30%	W14x61	Monotonic	Constant $P_g/P_{ye} = 0.3$
W-8-34-M-C-50%		Monotonic	Constant $P_g/P_{ye} = 0.5$
W-8-34-C1-C-50%		Collapse-consistent	Constant $P_g/P_{ye} = 0.5$
W-8-34-S-V-30%		AISC-symmetric	Varying $0.15 \leq P_g/P_{ye} \leq 0.75$
W-6-25-M-C-30%	W14x82	Monotonic	Constant $P_g/P_{ye} = 0.3$
W-6-25-M-C-50%		Monotonic	Constant $P_g/P_{ye} = 0.5$
W-6-25-S-C-50%		AISC-symmetric	Constant $P_g/P_{ye} = 0.5$
W-6-25-S-C-75%		AISC-symmetric	Constant $P_g/P_{ye} = 0.75$
W-6-27-M-C-30%	W16x89	Monotonic	Constant $P_g/P_{ye} = 0.3$
W-6-27-M-C-50%		Monotonic	Constant $P_g/P_{ye} = 0.5$
W-6-27-S-C-50%		AISC-Symmetric	Constant $P_g/P_{ye} = 0.5$
W-6-27-S-V-50%		AISC-Symmetric	Varying $0.25 \leq P_g/P_{ye} \leq 0.75$

From Table 1, two specimens from each one of the three sets is subjected to a constant compressive axial load ratio, P_g/P_{ye} = 0.3 and 0.5 (in which, P_g is the gravity load that is applied to the column and P_{ye} is the expected axial yield strength of the respective steel cross-section) coupled with monotonic lateral loading. These tests are useful to characterize the monotonic backbone curve of the steel columns. In order to investigate the effect of the lateral loading history on the steel column behaviour, the W14x61 column (noted as W-8-34-C1-C-50%) is subjected to a collapse-consistent loading protocol that represents the ratcheting behavior of a column in a steel MRF that approaches collapse [17]. In order to investigate the effect of high axial load demands on the steel column plastic deformation, the W14x82 and W16x89 test specimens are subjected to excessive axial compressive ratios P_g/P_{ye} = 0.5 (i.e., P_g/P_{cr} > 0.5, in which P_{cr} is the critical load of a column). Finally, in order to further investigate the differences of the hysteretic response between interior and end columns specimens W-6-27-S-V-50% and W-8-34-S-V-30% are subjected to varying axial load synchronized with the AISC symmetric lateral loading protocol [18] as shown in figure 1. Notice that a steel column can be subjected to high axial compressive loads (i.e., 0.75P_{ye}) as well as relatively high axial tensile loads (i.e., -0.15P_{ye}) after the gravity offset is applied.

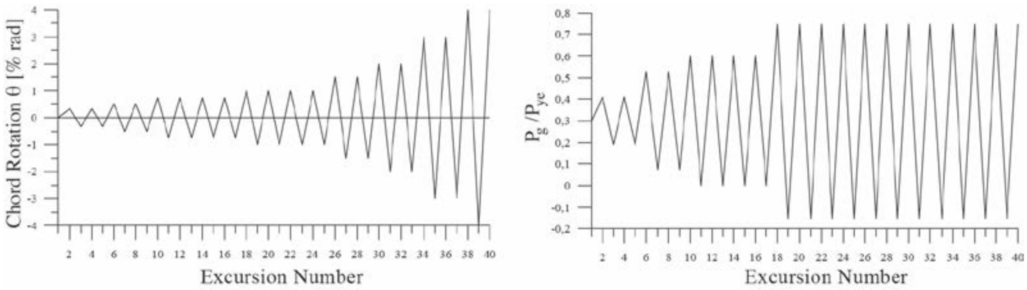

Figure 1. Dual parameter loading protocol for steel column experimental testing

2.1 Experimental setup for steel column large-scale testing

The experimental program is conducted at the Jamieson Structures Laboratory at McGill University with the test setup that is illustrated in figure 2. This setup consists of a high capacity vertical servo hydraulic actuator (11.4MN in compression/8MN in tension) that is utilized in force-control to apply the axial load demands on a test specimen. The axial load is transferred on a test specimen through an axially rigid link (see figure 2). A high capacity (±1000kN) / long stroke (±250mm) horizontal servo hydraulic actuator applies the lateral drift demands on a test specimen in displacement control. The reaction force from the test specimen is transferred to the test bed through a reaction frame that is pre-tensioned on the strong floor. A test specimen is subjected to unidirectional lateral loading. Out-of-plane deformations at the top end of a specimen are prevented through a set of guiding beams that are laterally supported by a specially designed reconfigurable out-of-plane lateral support system shown in red in figure 2.

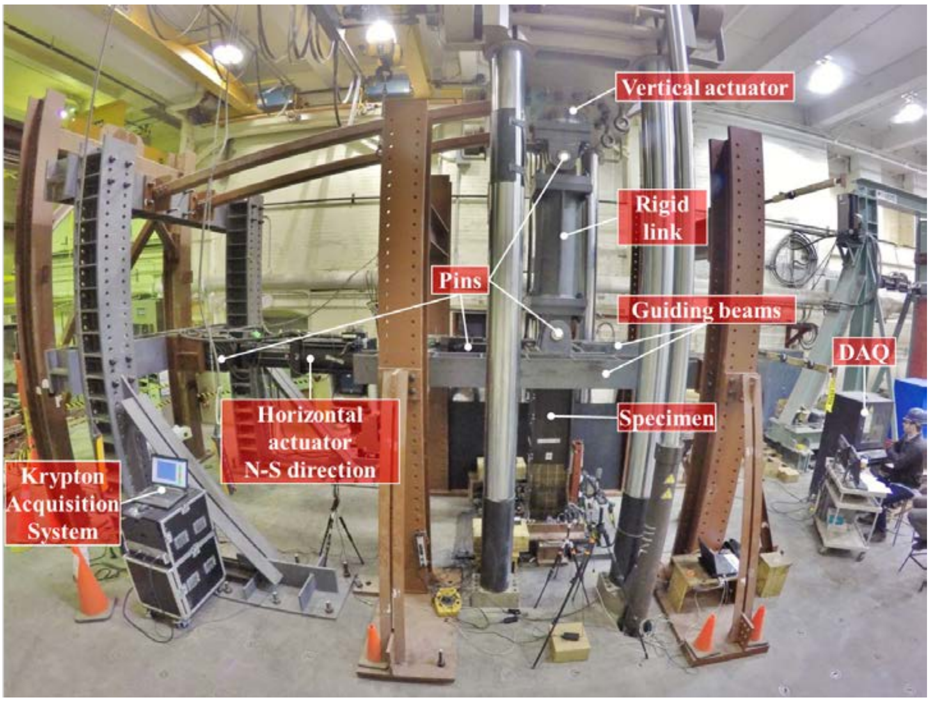

Figure 2. Test setup for large-scale steel column testing

3 EXPERIMENTAL RESULTS AND DISCUSSION

This section discusses preliminary findings from the experimental program outlined in Section 2. Emphasis is placed on the effect of axial load on the steel column performance, the differences in the observed response between interior and end columns due to varying axial load as well as the steel column stability. It should be noted that the hysteretic relations shown in the subsequent section include friction that has not been subtracted at the time of writing this paper.

3.1 Effect of axial load on steel column hysteretic response

Figures 3a and 3b illustrate the monotonic backbone curve for W14x82 and W16x89 steel columns respectively, for P_g/P_{ye} = 0.30 and 0.50. The observed differences in the flexural strength of the test specimens are due to the interaction of axial force and bending. The W14x82 steel column has a considerable plastic deformation capacity prior to the occurrence of local buckling even in the case of P_g/P_{ye} = 0.50 (see figure 3a). Similar findings hold true for the W16x89 steel column (see figure 3b). This curves can serve for the calibration of concentrated plasticity models (e.g., [19]) to facilitate the collapse assessment of steel frame buildings in seismic regions.

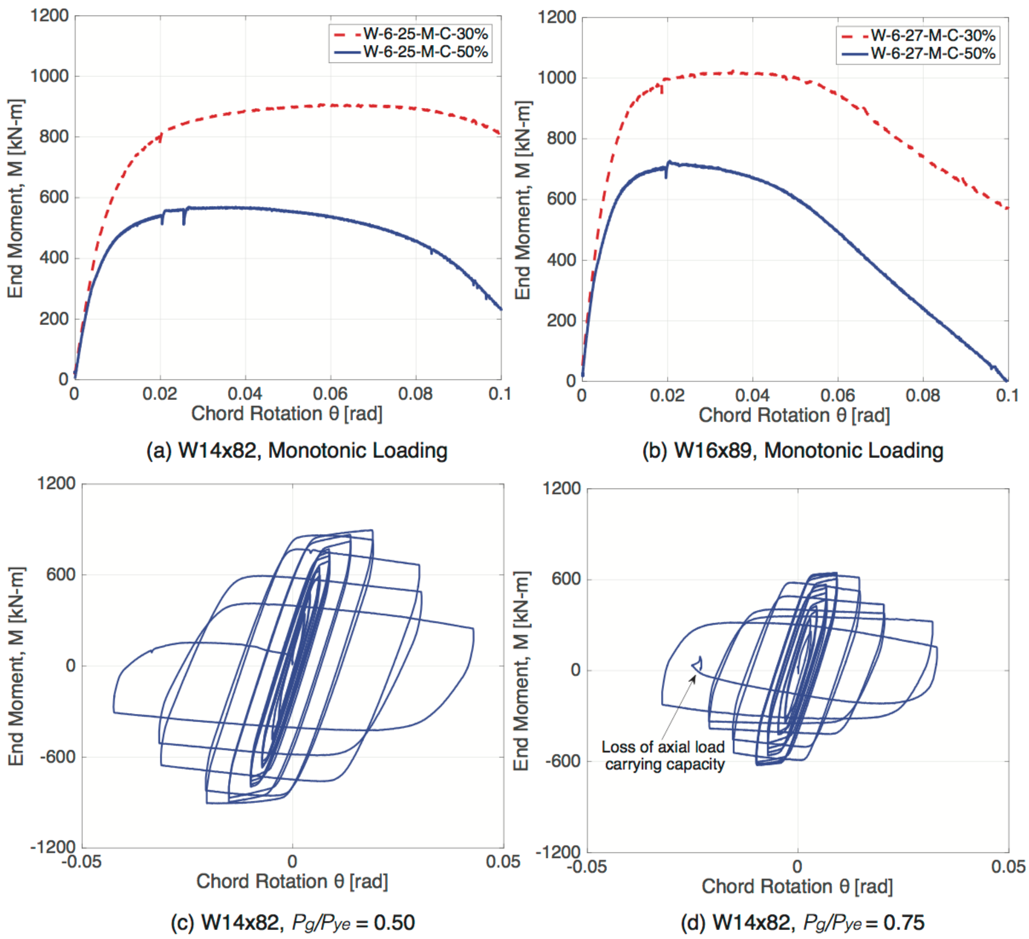

Figure 3. Monotonic and reversed cyclic moment-rotation relations for steel column specimens

Figures 3c and 3d illustrate the effect of the applied axial compressive load on the hysteretic behavior of nominally identical steel columns. Notice that in both cases, $P_g/P_{ye} \geq 0.50$. From these figures, the W14x82 steel column has appreciable plastic deformation capacity prior to the loss of its axial load carrying capacity even in cases that $P_g/P_{ye} = 0.75$. According to ASCE-41-13 [1] both cases would be treated as force-controlled elements; therefore, their plastic deformation capacity would be zero. From figures 3c and 3d, despite the presence of high axial compressive loads, both test specimens also developed considerable cyclic hardening prior to the onset of cyclic deterioration in flexural strength due to local buckling. This issue has direct implications in the employed strong-column-weak beam ratio that is typically used in the seismic design of steel MRFs [15,20].

3.2 Varying versus constant axial load

End columns in steel MRFs may experience fairly large axial load variation due to dynamic overturning effects during an earthquake. Figure 4 illustrates the column end moment rotation relation of two nominally identical test specimens that utilized a W16x89 cross-section and were tested under a symmetric lateral loading protocol coupled with a constant compressive axial load ratio P_g/P_{ye} = 0.50 (see figure 4a) and varying axial load demands ranging from $0.25P_{ye}$ to $0.75P_{ye}$ in compression. The gravity offset in this case is P_g/P_{ye}= 0.50. From figure 4, the hysteretic behavior of end columns is non-symmetric due to the variation of the applied axial load. In this case, the flange and web local buckling are straightened when the axial load varies from higher to smaller values. Because of the same reason, the in-cycle flexural strength deterioration of an end column is small (see figure 4b) compared to that of an interior column (see figure 4a). The post-peak slope of the moment-rotation relation of an end column may become fairly steep when the axial load increases from the gravity offset to a higher axial compressive load. The observed differences between interior and end columns are more pronounced when non-symmetric loading histories are employed as discussed in Suzuki and Lignos [5] or steel columns that utilize heavy cross-sections [21].

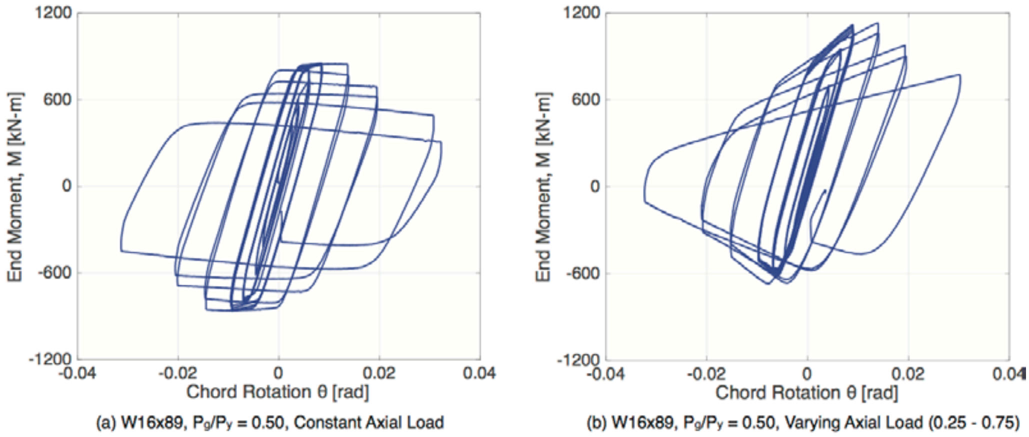

Figure 4. Constant versus varying axial load coupled with symmetric lateral loading history.

3.3 Steel column stability

Based on the damage progression observed in the test specimens, steel columns shorten axially after the formation of flange and web local buckling. This is due to the presence of the axial compressive load. This is a fundamental difference compared to the objected damage in steel beams as part of fully-restrained beam-to-column connections [22]. Steel column axial shortening is illustrated in Figure 5 for a W14x82 and a W16x89 column subjected to a constant compressive axial load ratio of 0.75 and 0.50, respectively, after the end of testing.

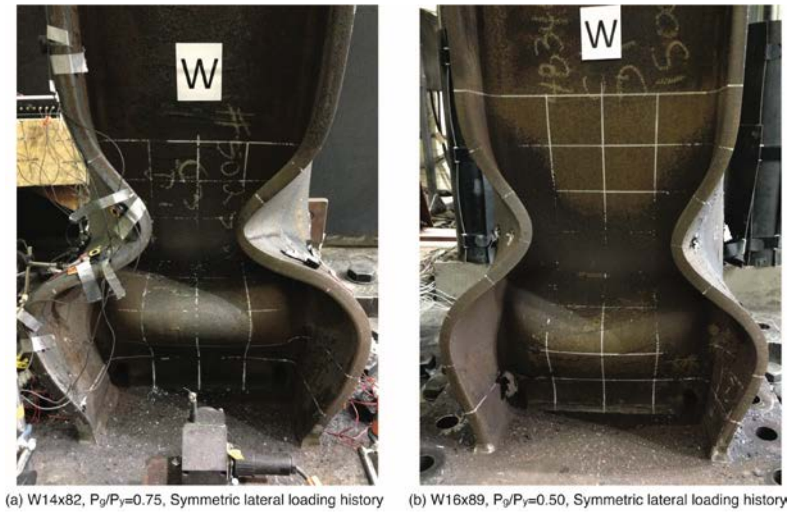

Figure 5. Column axial shortening and loss of axial load carrying capacity.

Figure 6a illustrates the typical progression of axial shortening with respect to the chord rotation of a W14x82 steel column subjected to a P_g/P_{ye} = 0.75 coupled with a symmetric lateral loading drift history. Based on this figure, the column axial shortening increases linearly prior to the onset of web and flange local buckling (i.e., up to about 1.5% rads). After this point, the column axial shortening increases exponentially. These observations are consistent with the ones discussed in MacRae [6] regarding the same failure mode. From figure 6a, once the column loses its axial load carrying capacity the axial shortening increases instantaneously for the same chord rotation. Note that the W14x82 steel column shortened by more than 9% of its height in this case. This is also illustrated in figure 6b that shows the applied axial load ratio P_g/P_y with respect to the column axial shortening for the same specimen. These results are consistent with the ones discussed in Suzuki and Lignos [5]. When the column loses its axial load carrying capacity axial force unbalance occurs. Once the force unbalance exceeds 25% of the applied compressive axial load on a steel column the vertical actuator is automatically switched from force to displacement control such that the test setup does not become unstable.

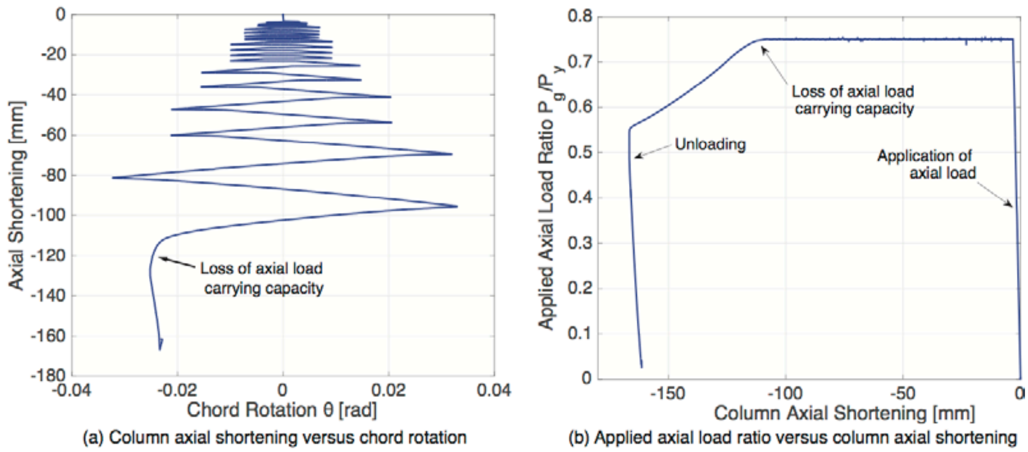

Figure 6. Column axial shortening and loss of axial load carrying capacity.

5 CONCLUSION

This paper summarized the preliminary findings of a large-scale experimental program that investigated the hysteretic behavior of wide-flange steel columns in steel moment-resisting frames (MRFs) under constant and varying axial load coupled with lateral drift demands. In order to challenge the existing nonlinear modelling recommendations for steel columns under high axial load ratios as per ASCE-41-13 [1] the emphasis of the experimental program was placed on the effects of high axial load demands on the steel column cyclic performance. For this reason, tests on three sets of four nominally identical test specimens were conducted. The specimens utilized W14x61, W14x82 and W16x89 cross-sections that were tested in a cantilever configuration with a fixed point of inflection. The main findings from the experimental program are summarized as follows:

- Slender but highly compact steel columns under high axial compressive load ratios (i.e., $P_g/P_{ye} \geq 0.50$) have a considerable plastic deformation capacity regardless of the employed lateral loading history. Therefore, the ASCE-41-13 [1] nonlinear modelling recommendations for force-controlled members should be revisited.
- End columns in steel MRFs are characterized by a non-symmetric hysteretic response in the positive and negative lateral loading direction. This is due to the variation of the applied axial load that the column experiences during the lateral loading history. Because of the same reason, the flexural strength of end columns deteriorates much slower compared to that of nominally identical interior columns.
- The steel column stability is characterized by axial shortening that increases linearly with respect to the lateral drift demands prior to the onset of flange and web local buckling. The axial shortening grows exponentially once the steel column cross-section buckles. Once a column loses its axial load carrying capacity due to severe axial and flexural strength deterioration, axial shortening grows instantaneously at a given lateral drift demand. It is suggested that this failure mode be considered as part of the seismic design of steel columns in MRFs.

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