Seismic Stability of Wide-Flange Steel Columns Interacting with Embedded Column Base Connections

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Abstract: The rotational flexibility of column base connections is commonly assumed to be detrimental to the seismic response of steel Moment Resisting Frames (MRFs) because it increases story drift demands and promotes soft-story formation by lowering the point of inflection, thereby exacerbating the moment demand at the first-story column top end. However, recent experimental research indicates that wide-flange steel columns with flexible boundary conditions may show enhanced response because base connection flexibility delays the plastic hinge formation and local buckling progression within the column cross-section near the base. Local buckling greatly reduces the lateral and torsional restraint at the column ends such that the delay in local buckling also delays member instabilities, which often control the deformation capacity of wide-flange steel columns. This effect is examined parametrically through 2160 Continuum Finite Element (CFE) simulations validated against physical experiments. Parameters interrogated include the applied axial load ratio, the column length, and the lateral loading protocol for the column sections, along with five levels of column base connection flexibility. The results indicate that base connection flexibility has a significant (and positive) influence on key aspects of column response, including: (1) lateral deformation capacity, which is on average 40~50% greater for a realistic value of flexibility, relative to a fixed-base condition, and (2) residual column axial shortening, which is important from the standpoint of retrofit and reparability (a 30-60% decrease). These benefits are more pronounced in cyclic (versus monotonic) loading histories and for members with cross-sections more prone to out-of-plane instability.

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The implications of these findings are outlined for existing buildings as well as prospective design methodologies. Limitations are discussed.

Keywords: Embedded Column Bases; Wide Flange Steel Columns; Boundary Conditions; Column Stability; Axial Shortening; Lateral-Torsional Buckling; Continuum Finite Element Simulation

Introduction

Steel Moment Resisting Frames (MRFs) in seismically active regions in the United States (AISC 2016a) and elsewhere (e.g., BCJ 2016; CEN 2004) are designed to concentrate plastic deformations in the beams while the columns should remain elastic under seismic shaking. To accomplish this, beams are designed for reduced seismic loads (i.e., using of the strength reduction factor, R), while column design is based on a combination of capacity design principles (i.e., Strong Column Weak Beam), as well as amplified seismic loads (i.e., the overstrength factor, Ω_0). The intent of this design practice is to discourage the development of soft-story mechanisms (Figure 1a) that concentrate yielding within one story, and to promote a full-building yielding sway mechanism (Figure 1b) in which beams from all stories participate. Despite its intent, this practice cannot eliminate plastic hinges in first-story columns in all cases, due to the following: (1) plastic hinges at the lower ends of first-story columns are a kinematic necessity for the formation of a full-frame mechanism as shown in Figure 1b, and (2) Strong Column Weak Beam design assumes a first mode response to determine the column strength demands; this is often violated, especially in mid- to high-rise frame buildings (Elkady and Lignos 2014, 2015b; Gupta and Krawinkler 1998) in which higher-mode response is likely to trigger soft story collapse mechanisms either in the first or upper stories of a steel MRF. In either case, the cyclic inelastic response of columns is of concern for two reasons. First, the plastic hinges weaken the column ends, rendering it susceptible to Lateral Torsional Buckling (LTB); this is particularly concerning since unlike beams, columns cannot be conveniently braced against LTB. Furthermore, steel MRF columns often carry substantial axial force and are in the vertical load path, such that their potential loss of axial load carrying capacity has severe implications for global collapse. As a result, the cyclic inelastic response of columns, with an emphasis on their cross-sectional and lateral-torsional stability and

the ensuing loss of lateral and vertical load carrying capacity has recently gained attention in research (Elkady and Lignos 2018a; b; Fogarty et al. 2017; Fogarty and El-Tawil 2016; Ozkula et al. 2017; Suzuki and Lignos 2015, 2018a; Wu et al. 2018). This research has yielded new information regarding the influence of various parameters (e.g., cross-sectional properties including width-thickness ratios, load histories, and boundary conditions) on the column's hysteretic response, deformation capacity, and lateral-torsional stability. A particularly interesting finding is that boundary conditions (specifically rotational fixities at the top and the bottom of the column in the major-axis direction) have a strong influence on lateral-torsional stability, and ultimately, the deformation capacity of the column. As an example, fixed-flexible columns tested by Elkady and Lignos (2018a) showed deformation capacities roughly 20-30% higher than their fixed-fixed counterparts – a significant difference in deformation capacity. As noted by Elkady and Lignos (2018), the physical mechanisms underlying this effect are somewhat complex and unintuitive. Specifically, the rotational flexibilities at the member ends control the moment diagram, which affects the plastic rotation, and more importantly, its apportionment between the top and bottom plastic hinge regions. In turn, the accumulated in-plane (major axis) plastic rotation at each plastic hinge region impacts its ability to provide out-of-plane, i.e., lateral-torsional restraint. Thus, the cyclic deformation capacity (corresponding to LTB) is controlled by highly nonlinear interactions between the hysteretic response of the plastic hinge regions (both in- and out-of-plane), the lateral stiffness of the column cross-section, and its resistance to lateral-torsional buckling.

From the standpoint of steel MRF design and collapse behavior, the above findings are relevant for two reasons: (1) moment frames are usually designed (and simulated) assuming an ideally fixed base boundary condition, entirely disregarding the effect of base flexibility on the MRF seismic response, and (2) recent research (Barnwell 2015; Grilli et al. 2017; Grilli and Kanvinde 2017; Rodas et al. 2017) has shown that even if designed as fixed, column bases of various types have considerable rotational flexibility. Zareian and Kanvinde (2013) have examined the “first-order” implications of such simplifications. These pertain to redistribution of moments in the first-story column due to the unanticipated base flexibility, leading to

moment amplification at the column top and the formation of a soft story. However, these studies do not consider the effect (discussed above) of base flexibility on out-of-plane stability of the column and associated change in deformation capacity – both of which are closely linked to system collapse (Lignos and Krawinkler 2012). Disregarding this consequential effect is problematic, especially as current guidelines for building performance assessment (FEMA 2009; PEER-TBI 2017) are heavily dependent on accuracy in collapse simulations.

Motivated by the above, this study presents parametric Continuum Finite Element (CFE simulations) to quantitatively examine the effect of boundary conditions (specifically base flexibility) on the cyclic inelastic response of wide flange steel columns; particularly their deformation capacity, axial shortening and lateral-torsional stability. The former is associated with a member's ductility, while the axial shortening and lateral-torsional stability are associated with the member stability. The study focuses on columns that respond as per the intent of current "strong-base-weak-column" practices (AISC 2016a), such that all inelastic response is concentrated in the column section, rather than the base connection. The main scientific basis of this study is a series of 2160 3-dimensional (3-D) CFE simulations that evaluate the following parameters: (1) column base rotational stiffness, (2) axial load ratio, (3) column length, (4) column cross-section, and (5) applied lateral loading protocol. The simulations use methodologies and calibration procedures that have been extensively validated against physical testing previously conducted by Elkady and Lignos (2015a, 2018a, 2018b), and the parameter ranges represent realistic values consistent with common design and construction practice.

The paper begins by summarizing recent research on columns, as well as column base connections that motivates this study. This is followed by a discussion of the FE simulations, including modeling assumptions/procedure, simulation matrix, and results. The main discussion of the results focuses on the interplay between boundary conditions and column properties to affect the cyclic deformation response and lateral-torsional stability of steel columns. A collateral finding (which pertains to the effect of boundary conditions on axial and lateral residual deformations of the columns) is also discussed; this is important

from the standpoint of post-event repair/retrofit strategies. The paper then examines the implications of this study in the context of improvements to design practice, and outlines future work necessary to achieve these improvements.

Full-scale Tests on Embedded Column Bases and Wide-Flange Steel Columns

Column base connections in steel MRFs may be broadly classified as being of the exposed base plate type (in which anchor rods attached to a base plate transfer flexure – see AISC Design Guide One (Fisher and Kloiber 2006)), or the embedded type, in which the lower portion of the steel column is embedded into a concrete footing. The former are usually feasible in low-rise MRFs where bases are assumed pinned, whereas the latter are more prevalent in mid- to high-rise MRFs (to carry large base moments), or when the bases are notionally fixed. Consequently, the latter (i.e., embedded type) connections are of interest in this study. Recently, Grilli et al. (2017) tested six large-scale specimens of embedded column bases subjected to combinations of axial (to represent gravity) and cyclic lateral (to represent seismic) loading. The specimens featured details common to United States construction practice (see Figure 2), including an embedded base plate at the bottom of the embedment, and a stiffener plate at the top of the connection to transmit axial compression. Referring to Figure 2, all specimens were tested as cantilever columns designed to be stronger than the connections, such that only connection failure modes were obtained. Referring to Table 1, which summarizes the test matrix from Grilli et al. (2017), a range of parameters (axial load ratio, column cross-section, and embedment depth) were interrogated. A similar experimental study was conducted by (Richards et al. 2018); albeit these specimens with somewhat shallower embedment were representative of exposed base connections with an overtopping slab – less relevant to the goals of this study, which focuses on bases with high notional rigidity.

Grilli et al. (2017) reported various failure modes (including concrete bearing failure ahead of the flanges and panel zone cracking, or uplift failure). Additionally, it was noted that all tested specimens showed significant rotational flexibility; Table 1 lists the values of the rotational stiffness for each of the specimens,

denoted β . As determined by Grilli and Kanvinde (2015), this degree of flexibility is of approximately the same magnitude as the rotational flexibility at the top of the first story column, which is the result of appropriately designed beams and the column above. This has two implications, relative to the fixed base assumption: (1) the point of inflection in the first story column moves downwards increasing moment demands at the column top end, while reducing moment demands at the bottom, and (2) the story drift increases appreciably (by approximately 30-40% - Grilli et al. (2017)). Zareian and Kanvinde (2013) examined these effects, and their influence on the system-level steel MRF response. However, this examination is restricted to in-plane effects and the associated frame response. The focus in this study is on out-of-plane effects – as illustrated schematically in Figure 3 at a given lateral drift demand in cases that the column base flexibility is considered (see Figure 3b) versus the idealized fixed column base (see Figure 3a). More specifically, the effect discussed in point (1) above results in a lowered inflection point, and consequently, a lower bending moment at the first story column base. In turn, this delays the formation of a plastic hinge at the column base, relative to the fixed-base configuration. It is well-known (e.g., Ozkula et al. 2017), that albeit the plastic hinge itself is associated with in-plane rotations, steel material inelastic behavior within the plastic hinge, limits its ability to restrain out-of-plane buckling modes such as LTB. As a result, despite their higher initial stiffness, columns with fixed bases are susceptible to local buckling at lower deformations as compared to similar columns with base rotational flexibility. In fact, a similar effect was noted previously by Elkady and Lignos (2018b), in which the flexibility at the column top end (due to the respective beam-to-column connection) increased the deformation capacity of the column with respect to member instabilities.

The foregoing discussion, when considered along with the findings of Elkady and Lignos (2018a; b) establishes that realistic values of column base flexibility have the potential to significantly impact the response (specifically the propensity for LTB) of wide-flanged columns in steel MRFs. The next section parametrically examines this effect through CFE simulations on a range of columns and base conditions that are realistic with respect to current United States design practice.

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149 **Continuum Finite Element Model for Steel Columns Interacting with Embedded Column Bases**

150 The seismic stability of wide-flange steel columns interacting with concrete footings is investigated by
151 means of CFE simulations. This section discusses the specifics of the CFE model including its validation.
152 This involves two steps. First the CFE simulations are validated rigorously by leveraging unique
153 experimental data from recent full-scale tests on wide flange steel columns (Elkady and Lignos 2018a) and
154 embedded column base connections (Grilli et al. 2017). Once validated, these are used to parametrically
155 comprehend the physical phenomena discussed earlier.

156

157 *Finite element model for wide flange steel columns*

158 The steel wide-flange column CFE model is developed in the commercial finite element analysis software
159 ABAQUS (Version 6.14-1) (ABAQUS 2014) based on the modeling recommendations summarized in
160 Elkady and Lignos (2015a, 2018a). Briefly, the CFE model consists of shell elements and considers material
161 nonlinearity through a multiaxial plasticity constitutive law (Lemaitre and Chaboche 1994; Voce 1948)
162 calibrated to uniaxial cyclic round coupon tests as discussed in Sousa and Lignos (2018). Residual stresses
163 due to hot-rolling are considered as discussed in Young (1972). Local and member instabilities are triggered
164 by assigning geometric imperfections that are within the allowable manufacturing limits (AISC 2016b;
165 ASTM 2017). The CFE modeling methodology is validated against experimental data from wide-flange
166 steel columns under multi-axis (i.e., lateral or bilateral coupled with axial) cyclic loading (Elkady and
167 Lignos 2018a). Table 2 summarizes key features of test specimens considered for the CFE model validation
168 including the applied column end boundary conditions, the applied constant compressive axial load ratio (P
169 is the applied axial load; $P_y = R_y f_y A$; R_y is the ratio of the expected yield stress to the nominal yield stress, f_y
170 is the nominal yield stress, and A is the cross-sectional area of the column) and the corresponding member
171 slenderness, L_b/r_y (L_b and r_y are the unbraced length and the radius of gyration about the column cross-
172 section's weak-axis, respectively).

Figures 4 and 5 illustrate comparisons of the column end moment, M_{base} versus the column drift, θ defined as the lateral displacement at the column top end over the actual column height, ($L' = L - \delta_v$, in which, L is the original column length and δ_v is the vertical displacement, if any), the evolution of the column axial shortening, respectively, obtained from the test results and CFE simulations. The second order moment, i.e., due to $P-\Delta$ effects, has been subtracted in all cases. The agreement between the test results and the CFE models across the boundary conditions, the unbraced length and the applied compressive axial load ratio indicates that the CFE models are able to reproduce the relevant aspects of physical response with high fidelity. As discussed by Elkady and Lignos (2018b), similar agreement between tests and CFE results is also noted between observed failure modes, such as local and lateral torsional buckling, and the deformations associated with them.

Base flexibility in column end boundary conditions

The CFE model is further refined to assess the influence of the column base flexibility on the overall stability of the steel column. The boundary conditions reflect those common in steel MRFs as shown schematically in Fig. 6. In particular, the column base flexibility is considered with an elastic rotational spring on the basis of the assumption that conventional column base design (AISC 2016a), which requires the base connection to be stronger than the column, results in plastic hinging in the column section above the foundation. The column base flexibility is deduced from full-scale ECB experiments (Grilli et al. 2017). These values are summarized in Table 1. The remaining degrees of freedom at the base are idealized as fixed.

Referring to Fig. 6, the column top end boundary conditions also require further attention to accurately capture structural response. In particular, a rotational spring is assigned with respect to the y -axis that characterizes the in-plane rotational stiffness of the girders intersecting the column by considering a strong column weak beam (SCWB) ratio of 1.3 that satisfies the capacity design requirements as per AISC (2016a) and CEN (2004); this spring considers the potential inelastic behavior of the girders. The out-of-plane flexibility due to typical shear tab beam-to-column connections (Liu and Astaneh-Asl 2000, 2004) and the

torsional resistance of the girders is idealized with a rotational spring assigned with respect to the x -axis, which is characterized by a trilinear hysteretic behavior as shown in Fig. 6. Finally, the torsional resistance due to bending of the shear tab plate, the torsional resistance of the second story column and the weak-axis bending resistance of the girders is aggregated into a third spring that is assigned at the column top end with respect to the z -axis. Specifically, the contributions of these three components is combined into a drift-dependent linear behavior. This behavior is inferred from experimental findings by Lignos et al. (2010) and Zhang and Ricles (2006) that represent typical response of fully restrained beam-to-column connections with Reduced Beam Sections (RBS).

Prior experiments on fully-restrained beam-to-column connections with deep columns (Chi and Uang 2002; Zhang and Ricles 2006) of similar sizes as the ones examined herein suggest that such columns experience torque demands after the local buckling formation within the RBS region. Referring to Fig. 7, this is attributed to the eccentricity e_x , which is introduced after local buckling initiation. Although the CFE model does not consider the full-frame, the aforementioned demands are still considered in the present study through a torque force – lateral drift relationship that is established on the basis of mechanics-based equations and experimental observations (Chi and Uang 2002; Lignos et al. 2010; Lignos and Krawinkler 2012; Zhang and Ricles 2006). This relationship is shown in Fig. 6. Referring to Fig. 7, Chi and Uang (2002) proposed the equation to estimate the torque force demand on the column, T :

$$T = F_{y,RBS} b_{f,RBS} t_{f,RBS} (e_x \cos \theta_{RBS} + e_y \sin \theta_{RBS}) \quad (1)$$

$F_{y,RBS}$, $b_{f,RBS}$, $t_{f,RBS}$ are the yield stress, flange width, flange thickness of the RBS region, respectively. e_x and e_y are the eccentricities of the beam flange force (F) due to beam LTB along each major axis; θ_{RBS} is the inclined angle of beam flange force due to beam LTB. All parameters except for e_x can be obtained from the geometry of beam/column and material properties of the beam; the e_x value in this study was deduced from the tests conducted by Zhang and Ricles (2006) in which the floor slab is also considered. Test results suggest that beam instability is induced at 2% lateral drift demand. The quantity e_x attains a value of 10% of the beam width at a lateral drift demand of 4%. The value of e_x increases while the local buckling

progresses within the RBS region. This relation is expressed through a linear function between the torque force-column drift relation. Two points are considered to establish this relationship (zero torque force at 2% and deduced torque force at 4%).

Parametric Finite Element Matrix

A parametric finite element study is established to assess the effect of column base flexibility on the overall steel column stability based on the proposed CFE model discussed earlier. Several parameters are interrogated. In particular, the column length covers the typical range of first story steel MRF columns (i.e., 3 to 6 meters). The employed cross-sections (18 in total) are shown in Fig. 8. Although the majority of these columns are within the limits of highly ductile members as per AISC 341-16 (AISC 2016a), moderately ductile cross-sections are also considered. The applied compressive axial load ratio, P/P_y , representing the gravity-induced axial load ratio that a first story steel MRF column experiences ranges from 5%, 20%, 35% and 50%. This range is deemed to be rational for low to high-rise steel MRFs designed in seismic regions (Suzuki and Lignos 2014, 2018b). Owing to the emerging need for data that characterize the monotonic behavior of structural components (Hamburger et al. 2016; Krawinkler 2009; Lignos et al. 2018), a monotonic lateral load is considered in addition to a standard symmetric lateral loading history (Clark et al. 2002). Finally, the considered cases are analyzed for various levels of column base flexibilities. Embedded column base experiments (Grilli et al. 2017) suggest that the “actual” flexibility of a column base connection designed to be stronger than the column may be assumed to be $\beta / (EI/L) = 5 \times (L / 4.0)$ (L in meters) (in which E is the modulus of elasticity of the steel material, I and L are the cross-sectional moment of inertia and member length of the column, respectively). This is considered to be the base case in the parametric CFE matrix, since it represents the most likely physical response. Supplementary parametric study simulations with base flexibilities 75% and 50% of the base case are also considered to examine the sensitivity of the column response to the base flexibility. The pinned column base is also considered for reference.

Finite Element Simulation Results and Discussion

General observations of physical response

This section presents key observations of the influence of the column base flexibility on the overall column stability under cyclic loading, based on the parametric CFE study outlined in the previous section. The discussion is facilitated by a comparison between the ideally ‘Fixed’ and ‘Actual’ case for a W24x146 column subjected to $P/P_y=20\%$, which is representative of all CFE simulations. Referring to Fig. 9a, the elastic stiffness of the column is reduced due to the actual flexibility of the column base. This can be explained by the elastic stiffness K^* equation (in $M_{base} - \theta$ relation) that considers all the flexibility contributions of a column member. The equation is derived by idealizing that the steel column is supported by two elastic rotational springs at each end boundary,

$$K^* = \frac{M_{base}}{\theta} = \frac{6EI}{L} \frac{\frac{2(EI/L)}{K_{top}} + 1}{\left(\frac{12(EI/L)^2}{K_{top}\beta} + \frac{4(EI/L)}{K_{top}} + \frac{4(EI/L)}{\beta} + 1\right) + \frac{12(EI/L)}{GA_w L} \left(\frac{EI/L}{K_{top}} + \frac{EI/L}{\beta} + 1\right)} \quad (2)$$

In which, A_w is the cross-sectional area of the column web; and G is the elastic shear modulus; K_{top} and β are the equivalent rotational stiffness of the springs at the column top and bottom, respectively; K_{top} may be deduced by the respective members connecting to the column top end (as discussed before by referring to Fig. 6), while β may be deduced based on the column base connection detailing as discussed in Grilli and Kanvinde (2015). Typical values for this quantity are listed in Table 1 for both shallow (W14) and deep (W18) column cross-sections. The CFE simulations suggest that although the elastic stiffness K^* decreases, the onset of the local buckling near the column base occurs at a larger drift ratio, thereby delaying the cyclic deterioration in the member’s flexural strength and stiffness.

Referring to Fig. 9b, the inflection point is at 70% and 60% of the column height for the ‘Fixed’ and ‘Actual’ cases, respectively. Although the inflection point of the ‘Fixed’ case is at a higher elevation than that of the ‘Actual’ case in the elastic regime, it moves downwards more rapidly due to the early onset of local buckling in the former than the latter, thereby increasing the flexural demands at the column top end. It is noteworthy

that after the 4% drift amplitude loading cycle, the inflection point location of the ‘Fixed’ and ‘Actual’ cases is at 45% and 55% of the member height, respectively.

The column base flexibility has a significant effect on the local buckling-induced column axial shortening as shown in Fig. 9c. In particular, the column axial shortening is appreciably reduced at a given drift level. For instance, at a 2% drift angle, typically associated with design basis seismic events, column axial shortening of the ‘Actual’ case is almost 50% less than the ideally ‘Fixed’ case. This has direct implications in the steel column reparability in the aftermath of earthquakes (Cravero et al. 2018) as it can be seen from Fig. 9d that shows the extent of local buckling near the column base for each one of the two considered cases. The total member rotation is disaggregated in Fig. 10 both for the fixed and Actual case. In the latter (Fig.10b), the column base flexibility reduces the accumulated plastic deformation in the column itself.

Influence of Column Base Flexibility on Base Moment vs Column Drift Response

Figures 11a and 11b show the load-deformation response (column base moment versus column drift) from CFE simulations of a monotonic load case (W24x146, $L=4500\text{mm}$, $P/P_y=20\%$, Monotonic loading) and a cyclic load case (W24x146, $L=4500\text{mm}$, $P/P_y=20\%$, Cyclic loading), respectively. The purpose of these plots (which are qualitatively representative of all CFE simulations outlined in the previous section) is to schematically illustrate key quantities retained from the CFE for the purpose of column performance assessment. These quantities have been determined to strongly influence the seismic performance and global collapse of steel MRFs (Elkady and Lignos 2014, 2015b; Ibarra and Krawinkler 2005, 2011; Lignos et al. 2011, 2013). Referring to Figs. 11a and b, these performance indicators include: (1) the peak moments M_{max} (M_{max}^*) where the term M_{max}^* in the parentheses denotes the peak (considering both directions) of the cyclic envelope, whereas the term M_{max} denotes the peak of the monotonic backbone (the two are different due to the influence of loading history and material cyclic hardening), (2) the deformation θ_{max} (θ_{max}^*), corresponding to the attainment of these peak moments, and (3) the “post-capping” plastic rotations θ_{pc} (θ_{pc}^*) corresponding to the rotation at which the moment carrying capacity is lost subtracted by θ_{max} (θ_{max}^*), indicating high potential for structural collapse (Ibarra and Krawinkler 2011). In cyclic loading, both

positive and negative excursions are considered to obtain these values. Figures 12a-c show monotonic curves for three different cross-sections (W36x529, W21x147, and W21x101), which are selected from the CFE simulation matrix outlined in the previous section, whereas Figures 13a-c show the first-excursion cyclic envelopes (i.e., a curve connecting the extreme points of each cyclic excursion) for the same three columns. In each case, the curves are plotted for four levels of base fixity ('Fixed', 'Actual', 75% 'Actual', 50% 'Actual'). The cross-sections are sampled from the test matrix to examine general trends across the web slenderness ratios (h/t_w). Referring to Figures 12 and 13, the following observations may be made:

- Referring to Figs 12a-c, it is readily apparent that as the cross-section local slenderness increases, the effect of base flexibility on the response increases as well. Specifically, for the W36x529 ($h/t_w = 21$, highly ductile), the curves from the four base fixities differ mainly (as expected) in the elastic region, but are virtually coincident as the response transitions to yielding, followed by attainment of the peak due to LTB, and post-peak response. As the local slenderness increases (Figures 12b and c), the curves from the different flexibilities slightly diverge. For the most slender section (W21x101), local buckling followed by LTB occurs at the end of the elastic phase which is longer for the more flexible base connection.
- Referring to Figures 13a-c, the effect of flexibility on the cyclic "first-excursion" envelope curves is significantly more pronounced as compared to the monotonic cases. For each of the three cross-sections examined in this figure, the drift θ_{max}^* corresponding to the peak increases as the column base flexibility increases. It is important to point out that this is not merely an artifact of the elastic stiffness being lower for higher base flexibility – since LTB typically follows after local buckling and it is attained well after yielding. More specifically, for each of the three sections, the increase in θ_{max}^* with respect to flexibility is mostly attributed to a delay in local buckling in the post-yield phase. Referring to the phenomenology of response discussed previously (see Figure 3), recall that LTB occurs when the torsional fixity at the ends is compromised due to formation of plastic hinges. As evidenced by Figures 13a-c and visual observations of response (e.g., Elkady and Lignos 2018a;

b), an increase in base flexibility delays the formation of the column plastic hinge at the base, and subsequently the loss of torsional fixity at the ends, resulting in improved column performance.

- Referring to Figs. 12 and 13, and the previous point, the effect of the column base flexibility is most pronounced for the rotation at peak, $\theta_{max} (\theta_{max}^*)$. In contrast, the peak moment, $M_{max} (M_{max}^*)$ and post-capping plastic rotation, $\theta_{pc} (\theta_{pc}^*)$ are relatively insensitive to the column base flexibility regardless of cross-sectional slenderness.
- Figures 14a and b plot θ_{max}^* for all the fixed base simulations against the ratio L/L_p (where L_p , which is indicative of the column resistance to LTB, is determined as per AISC 2016a). This provides a quantitative baseline of the deformation capacity of the column, against which the flexible base cases may be evaluated. Referring to Figures 14a and b, it is not surprising that θ_{max}^* decreases with L/L_p sharply dropping as L/L_p exceeds unity, indicating a transition from fully plastic response (attainment of M_p) to inelastic LTB in most cases. However, it is useful to interpret this in the context of the web slenderness ratio, h/t_w as discussed in the next section. Figures 14c and d indicate the deformation capacity for the flexible base conditions, normalized by the corresponding values (shown in Figures 14a and b) for the fixed base conditions. Figures 14a and c correspond to $P/P_y = 20\%$, whereas Figs 14b and d correspond to $P/P_y = 50\%$. Referring to Figs 14c and d, it is immediately apparent that (1) the base flexibility increases the deformation capacity with respect to the fixed condition, and (2) the degree of base flexibility controls this increase. More specifically, if the best estimate of base flexibility (i.e., ‘Actual’) is used in analysis, then the column’s deformation capacity is increased, on average, by up to about 40% and 50% for $P/P_y = 20\%$ and $P/P_y = 50\%$ with $L = 4500\text{mm}$, respectively – a significant increase. Further, using a base flexibility 50% of the base estimate (i.e., 50% ‘Actual’) results in an even greater (on average 75% and 97% for $P/P_y = 20\%$ and $P/P_y = 50\%$ with $L = 4500\text{mm}$, respectively) increase in the column deformation capacity. Implications of this finding in the context of design and performance assessment (along with associated trade-offs) are discussed in a subsequent section.

- The increase in deformation capacity due to the column base flexibility appears to show a positive trend with respect to L/L_p (Figure 14c), indicating that a greater benefit is obtained for members that are more susceptible to the out-of-plane instability. This is not surprising, considering the mechanism through which enhancement of deformation capacity occurs; specifically that the flexible base conditions limit the column inelastic rotation demands, thereby delaying the loss of out-of-plane restraint. This effect is reduced when (for the given length) the cross-section itself is highly resistant to out-of-plane deformations, e.g., for $L/L_p < 1$. No significant trend is observed with respect to the level of the applied axial load.

Influence of Column Base Flexibility on Column Residual Axial Shortening

Figure 15 shows the definition of column axial shortening, $\delta_{axial,\beta}$ considering the degree of column base flexibility, $\delta_{v,\beta}$ with reference to the undeformed, and pinned column case $\delta_{v,pin}$. Physical column testing (Elkady and Lignos 2018a; MacRae et al. 1990; Ozkula et al. 2017; Suzuki and Lignos 2015) complemented by CFE simulations (Elkady and Lignos 2015a, 2018b; Fogarty et al. 2017; Fogarty and El-Tawil 2016) suggest that column axial shortening is a primary failure mode currently not addressed in seismic code provisions; thus recent recommendations focus on how to limit column axial shortening in the context of seismic design provisions of steel MRFs (Elkady and Lignos 2018b; Wu et al. 2018), typically resulting in a heavier column design depending on the applied compressive axial load ratio. Recent work (Cravero et al. 2018; Suzuki and Lignos 2015, 2018a) suggests that column axial shortening may control decisions associated with reparability in the aftermath of earthquakes. In this section, the considered cross-sections are sampled to examine the influence of the degree of column base flexibility on the column residual axial shortening. The discussion is based on the CFE simulations under symmetric cyclic loading histories at story drift angles of interest to the engineering profession; namely, $\theta = 2\%$ associated with a design basis seismic event (i.e., 10% probability of exceedance over 50 years) and $\theta = 4\%$ associated with a maximum considered seismic event (i.e., 2% probability of exceedance over 50 years). Referring to Figs. 16 to 18, the following observations may be made:

- Figure 16a illustrates the strong influence of the column cross-section web local slenderness, h/t_w on the $\delta_{axial,\beta}$. The CFE simulations indicate that at 2% drift angle column axial shortening is up to 10mm, if the flexibility at the base is considered. While the assumed column base flexibility increases, the quantity $\delta_{axial,\beta}$ attains a value of nearly zero for the same reference drift angle. Cross-sections near the current seismic compactness classification limits as per AISC (2016a) shorten considerably when idealized as fixed. However, this varies depending on the assumed degree of the column base flexibility. For instance, at a reference drift angle of 2% the steel column with $h/t_w = 38$, $P/P_y=20\%$ and an intentional flexibility equal to 50% of the ‘Actual’ experiences nearly zero axial shortening compared to the ideally fixed case.
- Referring to Fig. 16b, the $\delta_{axial,\beta}$ is shown with respect to L/L_p at a reference drift angle of 4%. A $P/P_y=20\%$ is considered in this case for three different member lengths ($L=3000\text{mm}$, 4500mm , 6000mm). In the range of $L/L_p < 1.5$, columns with similar L/L_p values (i.e., similar r_y - radius of gyration with respect to the cross-section’s weak axis - for a given column length) result in a sudden increase of the column axial shortening as highlighted by the dashed lines. This suggests that column axial shortening is mostly controlled by the respective column web local slenderness given an axial load level because this is the only variable in this case. On the other hand, if $L/L_p > 1.5$, the column is mainly governed by out-of-plane deformation of the member, which does not result into excessive axial shortening (i.e., about 100mm at 4%) unless if the respective column utilizes a slender cross-section ($h/t_w > 30$ for $P/P_y = 20\%$).
- Past studies (Elkady and Lignos 2018a; b) highlighted the strong dependence of the column axial shortening with respect to the cumulative plastic rotation, the column web slenderness, h/t_w and the applied axial load ratio, P/P_y . Figure 17 shows the column axial shortening versus the cumulative plastic rotation relation for four different cross-sections (W36x194, W21x147, W24x146, W13x221) and four different degrees of column base flexibility. The assumed column length and applied axial load ratio were $L=4500\text{mm}$ and $P/P_y=20\%$ in all cases. The figure suggests that if the elastic contribution of the considered degree of column flexibility is subtracted then the column

axial shortening is nearly the same in all cases. However, this would occur at different lateral drift angles between cases. This indicates that the empirical equation proposed by Elkady and Lignos (2018b) may be used to compute the column axial shortening even in cases with intentional column base flexibility.

- To quantitatively support the above two points, Figure 18 shows the cumulative plastic rotation versus the column drift angle for different degrees of column base flexibility for a W24x146 steel column ($h/t_w=34$, $P/P_y=20\%$). Referring to Figure 18, the reduction of the cumulative plastic rotation from the fixed base case ranges from 30 to 50% depending on the assumed degree of column base flexibility. At larger drift angles, this reduction increases due to the delayed onset of local buckling for a given degree of column base flexibility.

Potential Implications for Seismic Design of MRFs

The parametric studies and the above discussion demonstrate the effects of base flexibility on various aspects of the cyclic response of wide-flange steel columns. Although these are interesting from a scientific and physical perspective, perhaps more importantly, they have major potential implications for the seismic design and performance assessment of steel MRFs. This is due to two factors: (1) these effects are quantitatively significant, such that key performance indicators vary by as much as 40~50% (for the capping rotation, which greatly affects collapse), and 30~40% for $P/P_y = 20\%$ and 50~60% for $P/P_y = 50\%$ (for the axial shortening, which affects retrofit decisions) in the comparison between cases with ideally fixed-base and actual embedded column base flexibility; such variation/error may be considered unacceptable, (2) the physical mechanisms and interactions responsible for these effects cannot be suitably modeled through frame simulations (and are complex to model even in CFE simulations), such that they are entirely disregarded within the current framework for seismic design and performance assessment. Against this backdrop, the following implications are noted for design and performance assessment:

- 423 • For existing buildings, whose collapse performance is of interest (e.g., within a PEER-TBI 2017 type
424 framework) in support of retrofit or insurance decisions, incorporating the beneficial effects of base
425 flexibility may have a significant (and positive) influence on the estimated probability of collapse, since
426 the effective deformation capacity of the columns is enhanced due to this flexibility. However, this
427 should be considered early on in the design phase so as the formation of first story collapse mechanisms
428 are prevented as discussed in the subsequent bullet point.
- 429 • For prospective design methodologies, the findings of this research possibly mitigate the problems
430 commonly associated with flexible bases. Specifically, it is well understood (e.g., Zareian and Kanvinde
431 2013) that even when capacity-designed to be stronger than the column, base connections have
432 appreciable flexibility; this lowers the point of inflection in the first story column triggering a soft-story.
433 Consequently, significant expense is often required to design the connections with high degree of fixity.
434 However, the current study indicates that such behavior appears in simulations only when the beneficial
435 effects of flexibility (i.e., by delaying local buckling) are not modeled. When these effects are modeled,
436 this negative influence of base flexibility is offset by the delay in local buckling; in fact, the net response
437 is improved with respect to the fixed-base case. The implication is that in many cases, the additional
438 expense required to provide base fixity may not be necessary, and may even be detrimental from the
439 standpoint of collapse performance.
- 440 • With reference to the above, it is relevant to note that there is interest within the professional practice
441 and research community (Castelo et al. 2017; Rodas et al. 2016; Trautner et al. 2016) to develop base
442 connections that are not capacity designed, such that they develop some degree of plastic deformation.
443 This too is motivated by the high expense of constructing strong base connections. A primary concern
444 in developing such lower-strength, ductile connections is their flexibility and its impact on the system's
445 seismic performance (since strength and stiffness are usually correlated). The research presented herein
446 assuages this concern to some degree.

- Recall that the beneficial effects of base flexibility are observed not only in the capping rotation, but also in the axial shortening. While important for retrofit decisions as indicated by a recent study (Cravero et al. 2018), assessment of axial shortening is currently not a part of established performance assessment frameworks (e.g., FEMA 2009; PEER-TBI 2017). As these frameworks evolve, it is anticipated that the findings of this research will gain even greater relevance.
- Although the study indicates the overall beneficial effects of base flexibility, previous findings (Zareian and Kanvinde 2013) suggest that to reduce the incidence of soft-stories (especially for first-mode dominated structures), strong-column-weak-girder provisions (AISC 2016a) may need to be revisited, to adjust the point of inflection to account for base connection flexibility. This is in line with recent studies related to the influence of the composite action on the collapse capacity of steel MRFs (Elkady and Lignos 2014).

In discussing these potential implications, it is important to emphasize that the research does not recommend design considerations, but rather indicates physical mechanisms and their quantitative effects that must be considered in support of such considerations. These mechanisms involving the interactions of local and LTB are inherently complex, may be incorporated either by CFE simulations (as done in this study), or by modifying hinge/frame element properties based on the functional relationships determined in this study (e.g., as shown in Figures 3, 9, 12, 13).

Summary and Conclusions

This paper presents the findings of parametric continuum finite element (CFE) simulations that investigated the hysteretic response, deformation capacity, and lateral-torsional stability of wide-flange steel columns embedded into concrete footings. The column base connections were considered to be capacity-designed (foundation designed to remain elastic and concentrate plastic deformation in the column) in accordance with the current design practice. The CFE simulations were based on a finite element model validated

against full-scale physical tests of steel columns. The CFE model was refined to represent the actual boundary conditions of columns in steel MRFs. In particular, the column base flexibility, β was considered based on measured values from prior embedded column base connection full-scale experiments. The column top end boundary was properly refined to represent the in-plane and out-of-plane flexibilities of the respective members intersecting the column. The torque demands imposed to the column after the beam-to-column connection plastification were idealized with a drift-dependent relationship established based on prior test observations. The CFE simulation matrix included variations in the applied compressive axial load ratio P/P_y , the imposed loading history (monotonic versus symmetric cyclic), the cross-section and member length, L as well as the assumed degree of column base flexibility, β .

As expected, the actual (versus fixed) column base flexibility reduces the lateral stiffness of the base moment – column drift relationship. In turn, this delays local buckling initiation (i.e., increases the rotation, θ_{max}^* or θ_{max} , at the peak load), thereby delaying the cyclic deterioration in flexural strength of the steel column as well as the initiation of axial shortening. If the best estimate of base flexibility is considered in the CFE model, then the column's deformation capacity at the peak moment is increased, on average, by up to about 40% for the expected range of compressive axial load ratios in first story steel MRF columns ($P/P_y \leq 0.20$). The CFE simulations indicate that a greater increase can be achieved in θ_{max}^* if a more flexible base is considered. The peak moment, M_{max}^* (or M_{max}) and post-capping plastic rotation, θ_{pc}^* (or θ_{pc}) are found to be insensitive to the degree of column base flexibility for all the investigated cases.

Wide-flange steel columns assumed designed to be weaker than the base connection, and therefore assumed to be ideally fixed (according to the current seismic design and simulation practice) experience considerable flexural strength deterioration at modest lateral drift demands ($\sim 1.5\%$ to 2%) when L/L_p exceeds unity (where L_p , which is indicative of the column resistance to LTB, is determined as per AISC 2016a). This is attributed to coupling of local and lateral-torsional buckling. On the other hand, the same columns with the true representation of the column base flexibility exhibit a positive trend with respect to L/L_p such that a greater benefit is obtained in members more susceptible to out-of-plane instabilities. An increase in the

column base flexibility delays the formation of column plastic hinging near its base, and subsequently the loss of torsional fixity at the column's ends. This benefit diminishes for columns with $L/L_p < 1$, with stocky cross sections that are less susceptible to lateral instability in the first place

The CFE simulations underscore the strong influence of the degree of column base flexibility on the resultant column axial shortening, which is an instability failure mode currently not addressed in seismic code provisions. In particular, steel columns with seismically compact cross-sections near the current classification limits as per the AISC (2016b) seismic provisions shorten considerably when idealized as fixed. This varies considerably depending on the considered degree of the base flexibility. At a reference lateral drift demand of 2% (indicative of a design basis earthquake) first story steel MRF columns with an assumed 50% 'Actual' degree of base flexibility attains nearly zero residual axial shortening. This may have direct implications in reparability of steel MRF columns in the aftermath of earthquakes.

The CFE simulations indicate that in the range of $L/L_p < 1.5$, columns with similar L/L_p values (similar r_y) experience a sudden increase of axial shortening. This implies that for a given compressive axial load ratio P/P_y , the web slenderness ratio, h/t_w primarily controls axial shortening. On the other hand, columns with $L/L_p > 1.5$ are mainly governed by out-of-plane deformations, which does not result in excessive axial shortening. The above findings are consistent with observations from physical testing on deep wide-flange steel columns (Elkady and Lignos 2018a; Ozkula et al. 2017).

The CFE simulations confirm given the column base flexibility and loading history, empirical expressions (Elkady and Lignos 2018b) provide good estimates of column axial shortening. This is because axial shortening (produced by irreversible local buckling deformations) is directly related to the cumulative plastic rotation demands in the column plastic hinge region. The present work indicates that embedded column base connections with some degree of flexibility relative to the fixed-base assumptions (either incidental - as in the case of existing buildings, or intentional – in the case of prospective buildings) improves the overall seismic stability, as well as reparability of wide-flanged steel columns. These potential benefits shall be interpreted in the context of the limitations of this study, which are now briefly summarized. First,

it is important to recognize that these effects have been observed only in CFE simulations and not system or sub-system tests with realistic boundary conditions – albeit the simulations themselves have been rigorously validated to capture the relevant physical phenomena. In fact, this study provides motivation for the experimental confirmation of these effects. Second, in this paper, the base flexibility is idealized as a single-degree-of-freedom rotational flexibility. In reality, connection details (e.g., reinforcement attachments to the embedment, thickness of embedded base plates etc.,) may influence other aspects of connection boundary condition such as warping restraint or translational flexibility. Finally, the analyses conducted in this study only examine the effect of in-plane loading, whereas first story columns in MRFs (even in planar frames) are often subjected to biaxial bending under multidirectional earthquake shaking. However, this limitation persists even in current simulation practice with fixed bases, wherein frames are seldom modeled in three dimensions. Future research is required to explore these limitations more fully. Notwithstanding these limitations, this study provides compelling evidence that if incorporated appropriately within current assessment techniques for performance based design (FEMA P695), simulation of the effects of column base connection flexibility offers significant potential benefits in terms of steel frame building performance.

Acknowledgments

This study is based on work supported by the Swiss National Science Foundation (Award No. 200021_169248). The financial support is gratefully acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of sponsors.

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Table 1. Summary of full-scale embedded column base connection test (Grilli et al. 2017)
 (*minus sign indicates tensile force)

Specimen ID	Cross- section	P (kN)	d_{embed} (mm)	$M_{max,ECB}$ (kNm)	β (10 ⁵ kNm/rad)
#1	W14x370	445	508	2613	3.23
#2	W18x311	445	508	2324	3.84
#3	W14x370	0	762	3741	3.07
#4	W14x370	445	762	4124	3.38
#5	W14x370	-667*	762	3800	3.25

678 **Table 2.** Test Specimens used for FE model validation (Test data from Elkady and Lignos 2018a)

Specimen ID	Cross- section	P/P_y	Boundary Conditions (Bottom-Top)	L_b/r_y	h/t_w	M_{max} (kNm)	EI/L (10^5 kNm/rad)
C1	W24x146	20%	Fixed-Fixed	51.7	33.3	2913	0.923
C2	W24x146	50%	Fixed-Fixed	51.5	33.1	2173	0.923
C3	W24x146	20%	Fixed-Flexible	51.5	33.5	2907	0.923
C7	W24x84	20%	Fixed- Flexible	79.2	47.0	1390	0.488

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681 **Table 3.** Simulation matrix (total 2160 cases)

Column base rotational stiffness, β	Length, L	Axial load ratio, P/P_y	Cross-section	Lateral loading protocol
5 cases (Fixed, Actual, 75%Actual, 50%Actual, Pin)	3 cases (3.0m, 4.5m, 6.0m)	4 cases (5, 20, 35, 50%)	18 cases (see Fig. 8)	2 cases (Monotonic, Symmetric cyclic)

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Figure Caption List

Fig. 1. Illustration of collapse mechanisms of MRFs: (a) Soft-story mechanism, and (b) full-frame yielding sway mechanism

Fig. 2. Details of typical embedded column base connections in United States

Fig. 3. Effect of flexible base on the cyclic response of steel columns (color indicates cumulative equivalent plastic strain) [W36x529, $L = 6000\text{mm}$, $P/P_y = 35\%$ ($h/t_w = 21$, $L_b/r_y = 59$)]

Fig. 4. FE model validation with full-scale test specimens in base moment – column drift relation: (a) C1, (b) C2, (c) C3, and (d) C7 (Test data from Elkady and Lignos 2018a)

Fig. 5. FE model validation with full-scale test specimens in axial shortening – absolute column drift relation: (a) C1, (b) C2, (c) C3, and (d) C7 (Test data from Elkady and Lignos 2018a)

Fig. 6. Boundary conditions and applied forces in developed finite element model

Fig. 7. Torque force demand on the column due to local buckling within the steel beam RBS region

Fig. 8. Selected steel column cross-sections for the parametric study

Fig. 9. Effect of the column base flexibility on (a) base moment – column drift relation, (b) inflection point – absolute column drift relation, (c) vertical displacement – absolute column drift relation and (d) deformed shape at 2nd cycle of the 2% column drift amplitude (color indicates cumulative equivalent plastic strain) in the representative case (W24x146, $L = 4500\text{mm}$, $P/P_y = 20\%$, $\beta = \text{Fixed or Actual}$) under cyclic loading

Fig. 10. Disaggregation of the column drift (W24x146, $L = 4500\text{mm}$, $P/P_y = 20\%$, Cyclic loading): (a) $\beta = \text{Fixed}$ and (b) $\beta = \text{Actual}$

Fig. 11. Schematic representation of primary column performance indicators: (a) Monotonic load case; (b) Cyclic load case

Fig. 12. Monotonic curves obtained from three different cross-sections with different column base flexibilities ($L = 4500\text{mm}$, $P/P_y = 20\%$, monotonic loading)

Fig. 13 First cycle envelope curves obtained from three different cross-sections with different column base flexibilities ($L = 4500\text{mm}$, $P/P_y = 20\%$, Cyclic loading)

Fig. 14. (a-b) Capping rotation, θ_{max}^* or (c-d) capping rotation, θ_{max}^* normalized by that attained in fixed base case, $\theta_{max,Fixed}^*$ vs column length normalized by the limiting laterally unbraced length, L/L_p relation under cyclic loading. Data from (a) Fixed base, $P/P_y = 20\%$, (b) Fixed base, $P/P_y = 50\%$, (c) $L = 4.5\text{m}$, $P/P_y = 20\%$, and (d) $L = 4.5\text{m}$, $P/P_y = 50\%$

Fig. 15. Definition of column axial shortening

Fig. 16. Column axial shortening vs (a) web local slenderness, h/t_w (expressed as function of column base flexibility) (data from $L = 4500\text{mm}$, $P/P_y = 20\%$, cyclic loading cases) and (b) column length normalized by the limiting laterally unbraced length, L/L_p (data from Fixed base, $P/P_y = 20\%$, cyclic loading cases, at 1st 4% positive excursion)

Fig. 17. Vertical displacement - Cumulative plastic rotation relation obtained from four different cross-sections (W36x194, W24x146, W21x147, W14x221) with different column base flexibilities ($L = 4500\text{mm}$, $P/P_y = 20\%$ case)

Fig. 18. Cumulative plastic rotation – column drift relation obtained from cases W24x146, $L = 4500\text{mm}$, $P/P_y = 20\%$ with different column base flexibilities

Fig. 1. Illustration of collapse mechanisms of MRFs: (a) Soft-story mechanism, and (b) full-frame yielding sway mechanism

Fig. 2. Details of typical embedded column base connections in United States

Fig. 3. Effect of flexible base on the cyclic response of steel columns (color indicates cumulative equivalent plastic strain) [W36x529, $L = 6000\text{mm}$, $P/P_y = 35\%$ ($h/t_w = 21$, $L_b/r_y = 59$)]

(a)

(b)

(c)

(d)

Fig. 4. FE model validation with full-scale test specimens in base moment – column drift relation: (a) C1, (b) C2, (c) C3, and (d) C7 (Test data from Elkady and Lignos 2018a)

(a)

(b)

(c)

(d)

Fig. 5. FE model validation with full-scale test specimens in axial shortening – absolute column drift relation:

(a) C1, (b) C2, (c) C3, and (d) C7 (Test data from Elkady and Lignos 2018a)

Fig. 6. Boundary conditions and applied forces in developed finite element model

Fig. 7. Torque force demand on the column due to local buckling within the steel beam RBS region

Fig. 8. Selected steel column cross-sections for the parametric study

Fig. 9. Effect of the column base flexibility on (a) base moment – column drift relation, (b) inflection point – absolute column drift relation, (c) vertical displacement – absolute column drift relation and (d) deformed shape at 2nd cycle of the 2% column drift amplitude (color indicates cumulative equivalent plastic strain) in the representative case (W24x146, $L = 4500\text{mm}$, $P/P_y = 20\%$, $\beta = \text{Fixed or Actual}$) under cyclic loading

757 **Fig. 10.** Disaggregation of the column drift (W24x146, $L = 4500\text{mm}$, $P/P_y=20\%$, Cyclic loading): (a) $\beta =$
 758 Fixed and (b) $\beta = \text{Actual}$

759

Fig. 11. Schematic representation of primary column performance indicators: (a) Monotonic load case; (b) Cyclic load case

(a) W36x529 ($h/t_w = 21$)

(b) W21x147 ($h/t_w = 28$)

(c) W21x101 ($h/t_w = 40$)

Fig. 12. Monotonic curves obtained from three different cross-sections with different column base flexibilities ($L = 4500\text{mm}$, $P/P_y = 20\%$, monotonic loading).

(a) W36x529 ($h/t_w = 21$)

(b) W21x147 ($h/t_w = 28$)

(c) W21x101 ($h/t_w = 40$)

Fig. 13 First cycle envelope curves obtained from three different cross-sections with different column base

flexibilities ($L = 4500\text{mm}$, $P/P_y = 20\%$, Cyclic loading).

(a)

(b)

(c)

(d)

Fig. 14. (a-b) Capping rotation, θ_{max}^* or (c-d) capping rotation, θ_{max}^* normalized by that attained in fixed base case, $\theta_{max, Fixed}^*$ vs column length normalized by the limiting laterally unbraced length, L/L_p relation under cyclic loading. Data from (a) Fixed base, $P/P_y = 20\%$, (b) Fixed base, $P/P_y = 50\%$, (c) $L = 4.5\text{m}$, $P/P_y = 20\%$, and (d) $L = 4.5\text{m}$, $P/P_y = 50\%$.

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