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REVISED ASCE-41 MODELING RECOMMENDATIONS FOR MOMENT-RESISTING FRAME SYSTEMS

D. G. Lignos[1], A. Hartloper[2], A. Elkady[3], R. Hamburger[4] and G.G. Deierlein[5]

ABSTRACT

Nonlinear static and dynamic analysis is utilized by engineers to evaluate the seismic behavior of new and existing structures in the context of performance-based earthquake engineering. Numerous experiments on steel moment-resisting frames and their components have been conducted over the past two decades. The findings from these tests suggest that the current ASCE-41-13 nonlinear component models do not adequately simulate the steel MRF component behavior. As part of the ATC-114 project, new modeling recommendations are proposed for several structural steel components of new and existing MRFs including, steel beams, columns, the beam-to-column web panel zone, column bases and column splices. These recommendations are based on a consistent methodology that takes advantage of unique experimental data as well as insights from detailed finite element analyses. For each structural component of interest a set of equations is developed to predict their first-cycle envelope and monotonic backbone curves that can be directly used in nonlinear frame analysis. The proposed equations also include information related to the associated modeling uncertainty of each of the input model parameters. Through an array of illustrative examples, it is shown that the new recommendations reflect much more accurately the behavior of structural steel components from the onset of damage through the loss of their load carrying capacity.

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[1]Associate Professor, Dept. of Arch. Civil & Environmental Engineering, Swiss Federal Institute of Technology, Lausanne (EPFL) (email: dimitrios.lignos@epfl.ch)
[2]Graduate Student Researcher, Dept. of Arch. Civil & Environmental Engineering, Swiss Federal Institute of Technology, Lausanne (EPFL)
[3]Post-Doctoral Researcher, Dept. of Arch. Civil & Environmental Engineering, Swiss Federal Institute of Technology, Lausanne (EPFL)
[4]Senior Engineer, Simpson Gumpertz & Heger Inc, San Francisco, CA
[5]Professor, Dept. of Civil & Environmental Engineering, Stanford University, Stanford, CA

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Nonlinear static and dynamic analysis is utilized by engineers to evaluate the seismic behavior of new and existing structures in the context of performance-based earthquake engineering. Numerous experiments on steel moment-resisting frames and their components have been conducted over the past two decades. The findings from these tests suggest that the current ASCE-41-13 nonlinear component models do not adequately simulate the steel MRF component behavior. As part of the ATC-114 project, new modeling recommendations are proposed for several structural steel components of new and existing MRFs including, steel beams, columns, the beam-to-column web panel zone, column bases and column splices. These recommendations are based on a consistent methodology that takes advantage of unique experimental data as well as insights from detailed finite element analyses. For each structural component of interest a set of equations is developed to predict their first-cycle envelope and monotonic backbone curves that can be directly used in nonlinear frame analysis. The proposed equations also include information related to the associated modeling uncertainty of each of the input model parameters. Through an array of illustrative examples, it is shown that the new recommendations reflect much more accurately the behavior of structural steel components from the onset of damage through the loss of their load carrying capacity.

Introduction

Practicing engineers adopted nonlinear structural analysis guidelines were adopted in performance-based design primarily after the publication of FEMA273/274 [1] and the ATC-40 guidelines [2]. The majority of commercial structural analysis software adopted the FEMA/273/274 nonlinear modelling recommendations and acceptance criteria that were based on available experiments at the time in an effort to utilize nonlinear analysis in design of new structures and the seismic retrofit of existing ones. The FEMA 273/274 guidelines were updated

[1]Associate Professor, Dept. of Arch. Civil & Environmental Engineering, Swiss Federal Institute of Technology, Lausanne (EPFL) (email: dimitrios.lignos@epfl.ch)
[2]Graduate Student Researcher, Dept. of Arch. Civil & Environmental Engineering, Swiss Federal Institute of Technology, Lausanne (EPFL)
[3]Post-Doctoral Researcher, Dept. of Arch. Civil & Environmental Engineering, Swiss Federal Institute of Technology, Lausanne (EPFL)
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and published as part of the ASCE/SEI 41-06 [3] and more recently the ASCE/SEI 41-13 [4] standards.

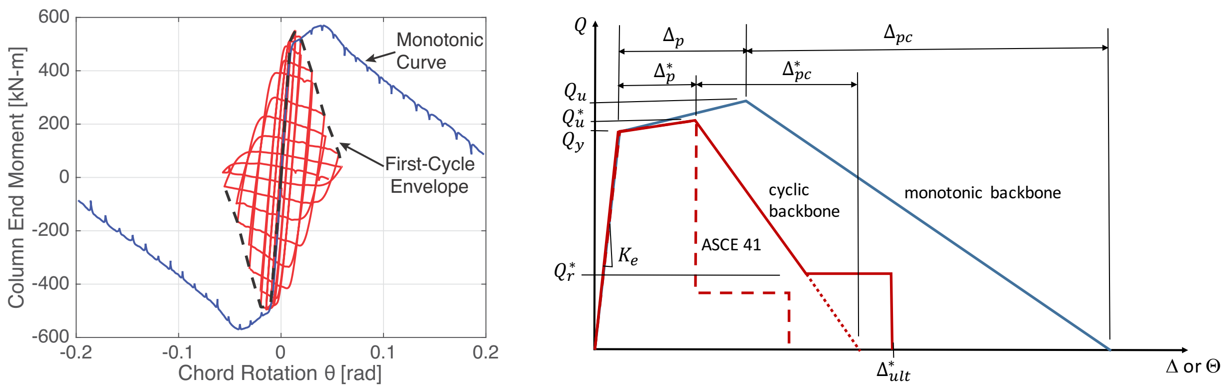
In the past 20 years, several experiments have been conducted to further our understanding regarding the hysteretic behavior of structural components. Despite the extensive experimental work, with few exceptions, the engineering profession mainly uses the basic relationships and acceptance criteria that were first developed and adopted in FEMA 273/274 Guidelines and Commentary [1]. The National Institute of Standards and Technology (NIST) identified the need to support further development of performance-based seismic design as a priority and sponsored the ATC-114 project, which is a series of projects to further this goal. The specific objectives of the ATC-114 project are to provide updated recommendations and acceptance criteria for nonlinear modeling of steel, reinforced concrete, wood and infill structures. The first author of this paper was appointed by the Applied Technology Council (ATC) as a technical committee member to develop updated modeling recommendations for structural components in steel moment-resisting frame (MRF) systems. The second and third authors were appointed by ATC as part of the working group associated with the same task. The fourth author served as the project director of the ATC-114 project and coordinated similar efforts related to all four steel materials. The last author served as a project director of a parallel effort that its goal was to develop guidelines for nonlinear structural analysis and design of buildings with steel moment frames (Part IIa). This paper provides an overview of the revised ASCE-41 recommendations for nonlinear modeling of steel MRF systems.

Definition of Monotonic Backbone and First-Cycle Envelope Curves

The proposed nonlinear modeling recommendations are developed on the basis of experiments conducted on structural steel components. Figure 1a shows an illustration of a steel column subjected to a monotonic and a symmetric cyclic loading history. The former is considered as a unique property of a structural component and can be used in nonlinear response history analysis if numerical models incorporate explicit rules for simulating component deterioration under cyclic loading. Referring to Figure 1a, the first-cycle envelope curve inherently traces the in-cycle strength deterioration that a structural component experiences; however, this curve is loading-history dependent [5]. The first-cycle envelope is meant to be used for nonlinear static analysis of new and existing buildings.

Figure 1b shows schematically the idealized multi-linear backbone curves of a structural component in comparison with the current ASCE-41-13 component curve. In brief, these curves are defined with an effective flexural stiffness, K_e; the effective yield strength, Q_y (i.e., M_y^* for most of the structural components discussed herein); the capping (peak) strength Q_u associated with the plastic deformation up to the peak strength, Δ_p (i.e., θ_p^* for the discussion herein); the residual strength, Q_r (i.e., residual flexural strength M_r); the plastic deformation of the descending portion of the respective backbone curve, Δ_{pc} (i.e., θ_{pc} for the discussion herein); and the ultimate plastic deformation, Δ_u (i.e., θ_u) that is associated with loss of the component's load carrying capacity.

The input deformation parameters of both the cyclic and monotonic backbones are fully defined based on empirical models that are developed from multiple regression analyses to the corresponding dataset of the component of interest. The variability of each parameter is expressed with a coefficient of variation (COV). These values can facilitate the development of appropriate demand and resistance factors in future editions of ASCE 7 [6]. Due to brevity, the subsequent sections provide a summary of the proposed recommendations for only key structural steel components with emphasis on the first-cycle envelope curves to be used in nonlinear static analyses of new and existing steel MRFs. However, modeling recommendations for nonlinear response history analysis have also been developed as stated earlier.



(a) Steel column (b) Idealized backbone curves

Figure 1. Definition of monotonic and first-cycle envelope curves of structural steel components (test data from [19]).

Steel Beams in Fully-Restrained Beam-to-Column Connections

This section provides a summary of the nonlinear modeling recommendations for non-composite and fully-composite steel beams in fully-restrained beam-to-column connections that meet the current ASCE-358-16 [7] requirements. Recommendations for beams in pre-Northridge beam-to-column connections are also provided.

Post-Northridge Beam-to-Column Connections

Steel beams that generally conform to AISC seismic design criteria in terms of lateral bracing, cross-sectional compactness, and are connected to steel columns such that the primary failure model is local buckling followed by ductile tearing due to low-cycle fatigue can be modeled based on Equations 1 to 3. Due to brevity, equations that refer to beams with a reduced beam section (RBS) are only shown herein. The equations have been developed for steel beams up to 36" deep based on a steel beam database assembled by Lignos and Krawinkler [8].

$$K_e = \alpha_e EI/L,\ M_y^* = \beta \cdot M_{pe},\ M_u^* = 1.1 \cdot M_y^*, (COV = 0.10),\ M_r^*, = 0.3 \cdot M_y^* \quad (1)$$

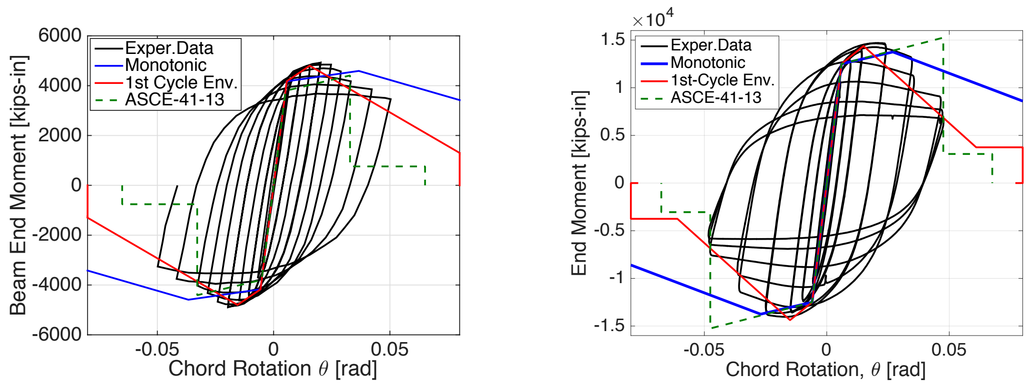
$$\theta_p^* = 0.55 \cdot \left(\frac{h}{t_w}\right)^{-0.5} \cdot \left(\frac{b_f}{2t_f}\right)^{-0.7} \cdot \left(\frac{L_b}{r_y}\right)^{-0.5} \cdot \left(\frac{L}{d}\right)^{0.8} (COV = 0.42) \quad (2)$$

$$\theta_{pc}^* = 20.0 \cdot \left(\frac{h}{t_w}\right)^{-0.8} \cdot \left(\frac{b_f}{2t_f}\right)^{-0.1} \cdot \left(\frac{L_b}{r_y}\right)^{-0.6} (COV = 0.31) \quad (3)$$

in which, a_e=60 by assuming the beam is in double curvature and that the plastic deformation is all concentrated in a point plastic hinge that is 10 times stiffer than the member flexural stiffness,

EI/L; β is a factor accounting for cyclic hardening effects and shall be taken as β=1.1 for beams with RBS; M_{pe} is the fully plastic bending resistance of a steel beam at the RBS region that is computed based on expected material properties and the reduced cross-section geometry; h/t_w is the web slenderness ratio; $b_f/2t_f$ is the flange slenderness ratio; L_b/r_y is the member slenderness; and L/d is the span-to-depth ratio.

Figure 2 shows a comparison between the proposed equations, the current ASCE/SEI 41-13 backbone and actual data from various steel beam geometries and connection topologies. It is shown that the M_u^*/M_y^* ratio is a more stable parameter than the constant 3% post-yield ratio to represent the post-yield component behavior. Although θ_p^* (i.e., "*a*" value of ASCE 41-13) is strongly dependent on the cross-sectional local slenderness ratios, the ASCE 41-13 component model tends to overestimate these values for deep beams (i.e., d > 21 inches). This is not the case for shallower beams (see Figure 2a). The θ_p^* of the first cycle envelope curve is, on average 0.65 times the initial backbone value based on monotonic loading, θ_p, which is consistent with the PEER/ATC-72-1 [9] modeling recommendations (i.e., Option 3 and Option 1).



(a) W18x40 beam with WUF-B (b) W30x99 beam with RBS

Figure 2. Comparisons of proposed modeling recommendations with ASCE/SEI 41-13 non-composite steel beams (data from Tsai and Popov [10]).

Equations 1 to 3 can be adjusted for fully composite steel beams. In particular, the effective yield flexural strength when the slab is in compression shall be adjusted based on section I3 of ANSI/AISC 360-16 [11]. It was also found that θ_p^{*+} under sagging is, on average, 1.8 times larger than the θ_p^* value of a non-composite steel beam.

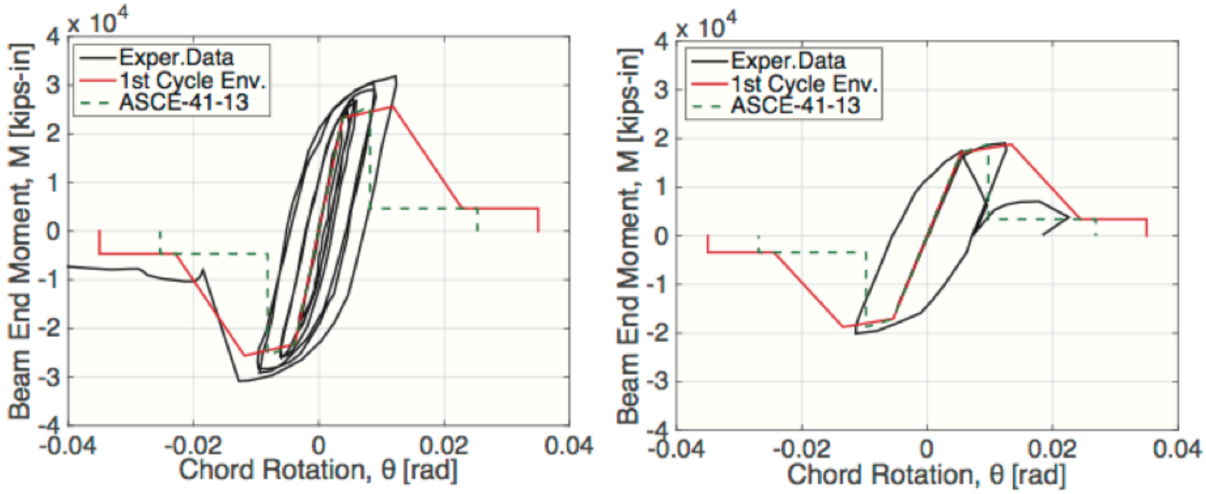
Pre-Northridge Beam-to-Column Connections

The θ_p^* value of steel beams with pre-Northridge beam-to-column connections with welded flange unreinforced web with a bolted shear tab (WUF-B), which was common in pre-Northridge steel construction, primarily depends on the beam depth, d and the respective yield ratio, F_t/F_y. The proposed equations for predicting θ_p^* and θ_{pc}^* are summarized below. The large COV values imply that these parameters vary considerably due to the nature of the observed failure modes (i.e., brittle).

$$\theta_p^* = \begin{cases} 0.046 - 0.0013d \geq 0 \ (COV = 0.50), \text{ if } d < 24" \text{ and } F_t/F_y < 0.6 \\ 0.008rads \qquad (COV = 0.64), \text{ if } d \geq 24" \text{ or } F_t/F_y \geq 0.6 \end{cases} \tag{4}$$

$$\theta_{pc}^* = \begin{cases} -0.003 + 0.0007d \geq 0 \ (COV = 1.1), \text{ if } d < 24" \text{ and } F_t/F_y < 0.6 \\ 0.035 - 0.0006d \qquad (COV = 1.1), \text{ if } d \geq 24" \text{ or } F_t/F_y \geq 0.6 \end{cases} \tag{5}$$

Figure 3 illustrates examples for steel beams in pre-Northridge WUF-B connections. The results suggest that the ASCE-41-13 component model seems to underestimate the pre-peak plastic rotation, θ_p^* of steel beams with d > 30 inches by 50%, on average. It was also found that for shallow beams (i.e., d < 21”) the proposed first-cycle envelope curve is nearly the same with the current ASCE-41-13 component model.



(a) W36x150 beam (WUF-B) (b) W36x150 beam (WUF-B)

Figure 3. Comparisons of proposed modeling recommendations with ASCE/SEI 41-13 for steel beams in pre-Northridge WUF-B beam-to-column connections (data from FEMA 1997 [12]).

Steel Columns

The proposed recommendations include two different sets of equations for modeling steel columns utilizing wide flange and hollow structural sections (HSS). The empirical equations are developed based on two steel column experimental databases [13-14] complemented with results from detailed finite element simulations [15]. Due to brevity, results for wide-flange steel columns are presented herein.

The effective stiffness, K_e of the proposed backbone models (see Figure 1) shall consider both flexural and shear deformations as proposed in Bech et al. [16]. The effective yield flexural strength M_y^* shall be computed based on the AISC interaction equations adjusted by 1.15 for the effects of material cyclic hardening (see Equation 6). The ultimate flexural strength, $M_u^* = a^* M_y^*$, in which $a*$ is a hardening parameter that is estimated by Equation (7) and depends on h/t_w, L_b/r_y and P_g/P_{ye} (in which, P_g and P_{ye} are the gravity-induced compressive axial load and the expected axial yield strength of the column, respectively). The θ_p^*, θ_{pc}^* values for wide flange steel columns are predicted by Equations (8) and (9), respectively. It was found that these parameters are primarily influenced by h/t_w because this controls column axial shortening [15]. In addition, in hot-rolled wide flange cross-sections there is a strong collinearity between $b_f/2t_f$ and h/t_w. The residual flexural strength at which local buckling stabilization occurs mainly depends on P_g/P_{ye} (see Equation 10).

$$\begin{aligned} &\text{If } P_g/P_{ye} \le 0.20, \quad M_y^* = 1.15 \cdot Z \cdot R_y \cdot F_y \left(1 - P_g/P_{ye}\right) \\ &\text{If } P_g/P_{ye} > 0.20, \quad M_y^* = 1.15 \cdot Z \cdot R_y \cdot F_y \cdot \left[\frac{9}{8}\left(1 - P_g/P_{ye}\right)\right] \end{aligned} \tag{6}$$

$$a^* = 9.5\left(\frac{h}{t_w}\right)^{-0.4}\left(\frac{L_b}{r_y}\right)^{-0.16}\left(1-\frac{P_g}{P_{ye}}\right)^{0.2} \ge 1.0,\ (COV = 0.07) \tag{7}$$

$$\theta_p^* = 15\left(\frac{h}{t_w}\right)^{-1.6}\left(\frac{L_b}{r_y}\right)^{-0.3}\left(1-\frac{P_g}{P_{ye}}\right)^{2.3} \le 0.10, (COV = 0.31) \tag{8}$$

$$\theta_{pc}^* = 14\left(\frac{h}{t_w}\right)^{-0.8}\left(\frac{L_b}{r_y}\right)^{-0.5}\left(1-\frac{P_g}{P_{ye}}\right)^{3.2} \le 0.10, (COV = 0.40) \tag{9}$$

$$M_r^* = \left(0.4 - 0.4\frac{P_g}{P_{ye}}\right)M_y^* \quad (COV = 0.35) \tag{10}$$

Figure 4 shows illustrative comparisons for both monotonic and reversed cyclic test results on wide flange steel columns subjected to a range of axial load ratios, P_g/P_{ye}. The findings suggest that the proposed first-cycle envelope curve represents relatively well the measured response of the wide flange steel columns regardless of the cross-sectional local slenderness ratio and the applied compressive axial load ratio. Figures 4c to 4e suggest that the ASCE-41-13 component model overestimates by a considerable amount the plastic deformation, θ_p^* of wide flange steel columns subjected to P_g/P_{ye} = 0.20 and 0.30. Unlike the proposed recommendations, the ASCE-41-13 model does not capture the cross-section local slenderness effects on the parameter "a". Figure 4f suggests that steel columns that utilize cross-sections within the limits of highly ductile members as per AISC-341-16 [17] and subjected to P_g/P_{ye} = 0.50 (i.e., $P/P_{cl} > 0.50$) have an appreciable plastic deformation capacity that is significantly underestimated by the current ASCE-41-13 provisions, which treat such columns as force-controlled elements. Instead, the limit for force-controlled elements shall be raised to $P_g/P_{ye} \ge 0.60$.

Experiments [13] suggest that for all practical purposes the plastic deformation capacity of wide-flange steel columns subjected to biaxial bending is the same with those subjected to unidirectional bending. However, M_y^* shall be reduced for the effects of biaxial bending in this case. Similarly, experimental work associated with end (exterior) columns [19] that experience axial load variation demands due to dynamic overturning effects indicates that their θ_p^*, θ_{pc}^* values shall be computed based on the gravity induced compressive axial load ratio, P_g/P_{ye}. Therefore, the same equations presented above should be utilized.

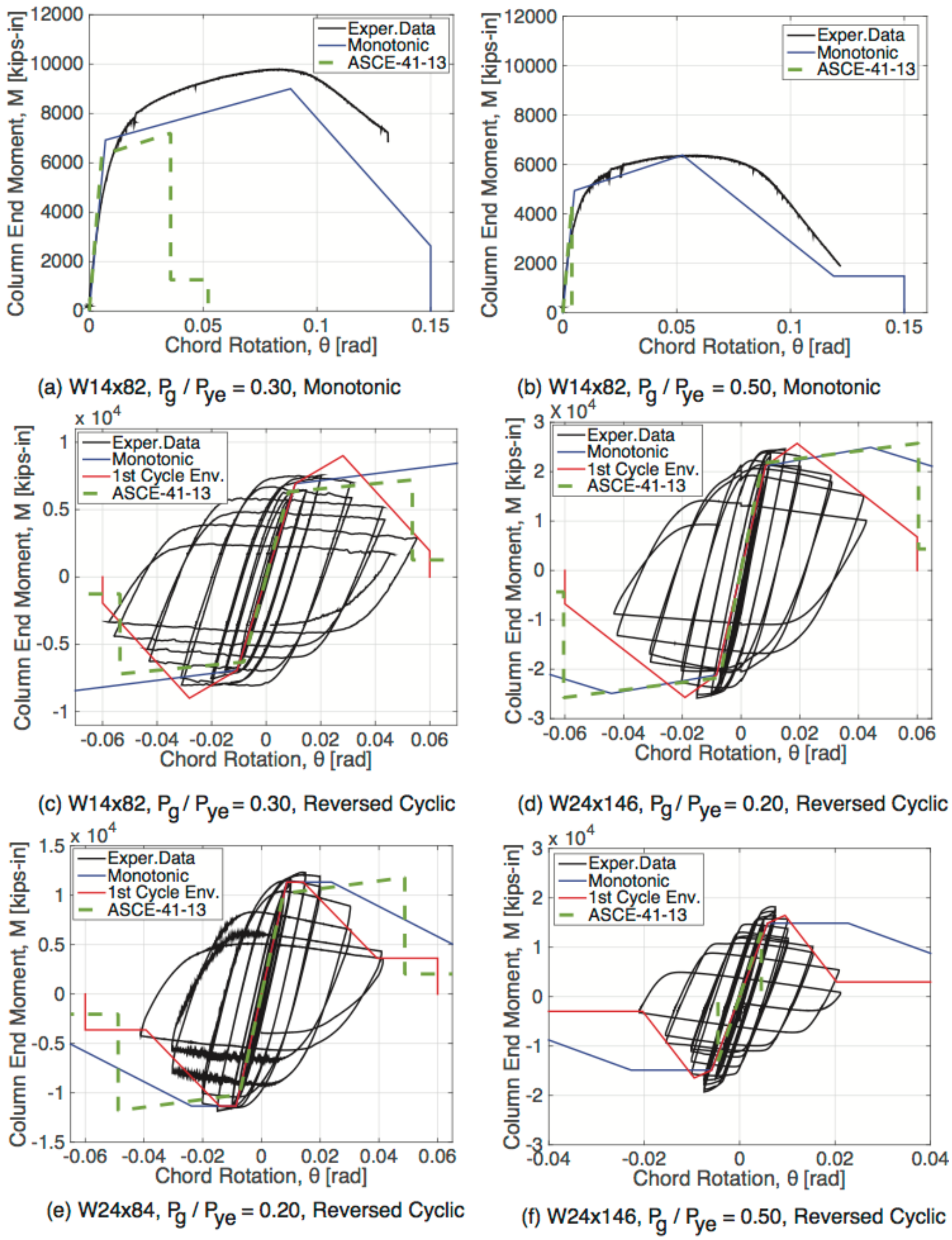


Figure 4. Comparisons of proposed modeling recommendations with ASCE/SEI 41-13 for wide-flange steel columns (data from [13, 18, 19]).

Other Structural Steel Components

Modeling recommendations are provided for a number of other structural steel components of steel MRF systems including beams in shear-tab beam-to-column connections, beam-to-column web panel zones, exposed and embedded column base connections as well as column splices. The recommendations are not included herein due to brevity. Detailed summaries regarding these recommendations can be found in [20]. Figure 5 shows an illustrative comparison of the proposed first-cycle envelope curve for exposed column base connections tested by Kanvinde et al. [21].

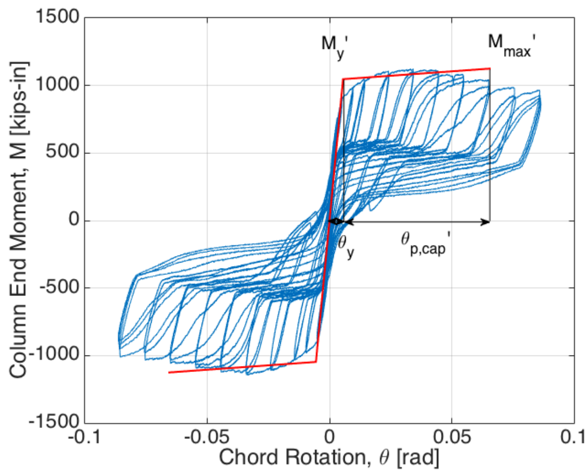


Figure 5. Comparisons of proposed modeling recommendations for exposed column base connections (data from [21]).

Conclusions

This paper provides a comprehensive summary of the revised ASCE 41 modeling recommendations for new and existing steel moment-resisting frame (MRF) systems based on the recently completed ATC-114 project in support of nonlinear analysis in design and retrofit. In particular, a set of comprehensive recommendations is provided regarding improved backbone relationships for structural steel components in MRF systems. The relationships reflect the trends observed from recent experiments that investigated the hysteretic behavior of various structural steel components. Two sets of relationships are proposed. The first one defines the idealized first-cycle envelope curve of structural steel components to be used in nonlinear static analysis. The second one defines the monotonic backbone curve, which is considered to be a unique property of a structural component. This is meant to be used for nonlinear response history analysis with numerical models that explicitly simulate deterioration characteristics (i.e., strength and stiffness) of various structural steel components. The recommendations provided herein were also adopted in a companion project that was conducted under the ATC-114 contract to address detailed modeling and analysis criteria for structural steel moment frames [22].

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