Behavior of Deep, Wide-Flange Steel Beam-Column Members in Seismic Applications

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ABSTRACT

This study involves a parametric analytical investigation of the behavior of deep columns with one-sided reduced beam section (RBS) connections for application in special moment frames (SMFs). Earlier studies led to the prequalification of RBS moment connections for column sizes up to W14 sections; however, the use of deeper columns in SMFs would be advantageous because of their ability to economically control drift. Information on deep column behavior using an RBS moment connection is limited, and this study investigates this behavior using a total of 40 assemblies designed according to the 2016 AISC *Seismic Provisions*. Four column sections were investigated—W14×426, W24×192, W27×194 and W30×191—each subjected to five levels of axial load, two levels of panel zone strength, and modeled conservatively without floor slab restraint. The results show that although the twist of the column increases with increasing column depths, all assemblies subjected to load below the column's design axial capacity still exhibited plastic hinge formation in the RBS. Additionally, the results show that for each column section investigated, the magnitude of twist decreases with an increase of the axial load on that section. Results also show that columns fitted with a doubler plate twist more than the corresponding configurations without a doubler plate. The study concluded (1) that increased column depth does not have a negative impact on the behavior of the connection as long as the axial loads in the columns are below 80% of the design capacity and (2) that deep columns can be considered as a valid alternative to W12 and W14 sections that are commonly used for RBS connections in SMFs, as long as they are properly detailed.

Keywords: deep columns, beam-columns, SMF, RBS, panel zone strength.

INTRODUCTION

After the Northridge earthquake in 1994, where damage in welded moment-resisting connections was discovered, numerous new beam-to-column connection details for steel special moment frames (SMFs) were developed (Ricles et al., 2004). One of these was the reduced beam section (RBS) connection, where portions of the beam flanges are trimmed away near the beam ends. The RBS connection forces yielding to occur in the reduced section of the beam, away from the face of the column, which contributes to ensuring the strong column–weak beam behavior that is required for SMFs (Engelhardt et al., 1996).

The SAC project, which investigated the performance of steel moment-resisting frame connections after the Northridge earthquake, mostly focused on the experimental testing of RBS connections with W12 and W14 columns (FEMA, 2000). Columns that are generally stocky are shown to perform well under seismic loading because they preserve their structural integrity at large inelastic deformations (NEHRP, 2011). However, deeper columns are more effective and economical at controlling drift in SMFs but can be susceptible to undesirable buckling behaviors (NIST, 2016). Several deeper column sections were tested in the SAC project, but the tests showed that stability problems occurred when RBS connections were used with deep columns (FEMA, 2000). Weak-axis and local buckling failure modes play a prominent role on deeper, more slender columns, and therefore the test results and conclusions reached based on stocky columns may not be generally applicable. Published research on the behavior of connections to deep, slender columns is limited, thereby requiring a more thorough body of work as was begun recently through a multiyear, federally funded research effort (NEHRP, 2011).

Due to their flexural stiffness, deep columns can effectively control drift in special moment frames (SMFs) while potentially yielding significant cost efficiencies over commonly used stocky column sections. Earlier research found that using a W27 column section in lieu of a W14 column section could result in a material savings of 6 to 8% for a typical 10-story building (Shen et al., 2002).

Because existing research on the behavior of connections to deep columns is very limited, this study presents a parametric analytical investigation of the behavior of deep

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columns with a one-sided RBS connection for applications in an SMF. The geometry and boundary conditions of the analytical model were based on experimental tests by Engelhardt et al. (1998) that used a one-sided RBS connection. A total of 40 assemblies were designed according to the 2016 AISC *Seismic Provisions* (AISC, 2016b). Column sections investigated were W14×426, W24×192, W27×194 and W30×191. The columns were subjected to five levels of axial load and had medium or weak panel zone strength. For this study, performed at the University of Cincinnati (Pettersson, 2015), the finite element software ABAQUS (Simulia, 2014) was used to develop high-definition finite element assemblies, which were subjected to a constant vertical load and a monotonically increasing shear/moment combination on the connected beam. The dimensions of the RBS cut complied with the requirements contained in AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2016a), and the assemblies did not include the lateral restraint provided by a floor slab.

The main objectives of the study are to (1) evaluate the effect of column depth on the seismic performance of connections in SMFs using RBS connections; (2) evaluate the effect of panel zone strength on the seismic performance of RBS connections to deep columns in SMFs, focusing on weak and balanced panel zone strengths; and (3) evaluate the effect of various levels of axial loads on the seismic performance of RBS connections using deep columns in SMFs.

Only connections to the strong axis of the column were included in this study, using RBS connections with a W36×150 beam in each case. The impact of residual stresses on column twist was also investigated for cases with and without doubler plates in column sections.

PREVIOUS RESEARCH

At the University of California, San Diego, Gilton et al. (2000) studied how the performance of RBS moment connections was affected by column depth in experimental and analytical studies. Three specimens were tested with one-sided connections without any floor slab. Two of the tests were ended prematurely because of the twisting and out-of-plane bending of the column. These specimens reached 0.03 radian of plastic rotation without any fracture in the welds before the tests were stopped. During the third test, a brittle fracture developed along the k-line of the column just before a plastic rotation of 0.03 radian was reached. All tested specimens experienced twist in the column, and in order to reduce the torsion of the column, it was recommended that bracing should be provided for the column or near the RBS region in the beam.

Shen et al. (2002) analytically investigated the use of deep columns in SMFs. Their work focused on two major parts. The first part was to conduct pushover analyses of two frames and compare their seismic behavior, where one frame used W14 column sections and the other used W27 column sections. The second part was to analytically reproduce the experiment by Gilton et al. (2000) and investigate the effects of different column depths. Shen et al. also expanded the study to models that used more realistic boundary conditions than those used by Gilton et al. Conclusions from the study were that deep column connections should be able to provide the strength required for prequalified connections if realistic boundary conditions were used, including accounting for lateral and torsional supports of columns at the floor level, provided by floor systems. The authors concluded that the cyclic behavior of an RBS connection to a deep column was similar to the behavior of the same connection with a W14 column when a floor slab was provided on at least one side of the beam. Altogether, the study by Shen et al. could not find any reasons to prevent the use of deep columns in moment frames.

At Lehigh University, Ricles et al. (2004) analytically and experimentally investigated the effect of column depth on the seismic behavior of both cruciform and one-sided RBS connections to deep columns. The project showed that when a floor slab is present, or when adequate lateral bracing is provided, both the twisting of the deep columns and the lateral movement of the beam in the RBS are reduced, and good performance can be achieved with deep columns if the column section satisfies the strong column–weak beam criterion. Other conclusions from the project included the importance of the panel zone strength, where a weaker panel zone increased the potential for a ductile fracture of the connection while stronger panel zones increased the column twist. The authors recommend a balanced panel zone, which can be achieved by using the AISC *Seismic Provisions*. The authors also studied the effect of axial load on the connection behavior, noting the axial load had a very small effect on global behavior and local fracture potential.

In 2006, Newell and Uang (2006) published a report on a series of experimental tests and ancillary numerical analyses on the cyclic behavior of steel columns subject to high axial loads and large drift demands. The finite element analyses indicated that significant local buckling occurred in W27×146 and W27×194 columns at 5% drift under three axial loading schemes ($0.35P_y$, $0.55P_y$, $0.75P_y$). The authors concluded that the cyclic response of W27 columns under high axial loads shows considerably more degradation than W14 columns.

FINITE ELEMENT ANALYSIS

Model Validation

This study used a variation on the modeling procedure presented by Ruffley (2011). This procedure calls for the use of eight-node, solid brick elements with reduced integration

Table 1. Material Data			
Member	Material	Yield/Ultimate Stress (ksi)*	Plastic Strain
Column	A572-50	49.9 (y)	0
		74.5 (u)	0.125
Beam W36×150	Unknown	41.4 (y)	0
		58.7 (u)	0.190
Continuity plate	A572-50	52.3 (y)	0
		52.6 (p)	0.00530
		85.9 (u)	0.119
Shear tab PL⅜×6×30	A572-50	52.3 (y)	0
		52.6 (p)	0.00530
		85.9 (u)	0.119
Doubler plate	A572-50	52.3 (y)	0
		52.6 (p)	0.00530
		85.9 (u)	0.119

* (y) = yielding; (p) = stress at the end of plateau; (u) = ultimate stress.

and hourglass control to create a fine mesh in the connection area up to twice the beam depth away from the connection. Ruffley suggested the use of a progressively coarser mesh for the rest of the length of the beam, as well as for the column away from the panel zone. Considering that this study focuses on the influence of the behavior of the column on the connection response, a finer mesh was created to model the column away from the panel zone as well so that buckling, stress concentrations, and other localized phenomena could be better predicted. To validate the modeling procedure, one of the specimens tested by Engelhardt et al. (1996) was replicated using ABAQUS. Specimen DB2 consisted of a 134-in.-long W36×150 beam connected to a 136-in.-long W14×426 column with a one-sided RBS connection. The material properties reported by the authors were incorporated into the finite element model and are summarized in Table 1. In particular, the yield stress values used for the beam and the column members were recorded in the experiments by Engelhardt (1998), and the remaining material data used in the model were those obtained from Ruffley (2011) as a curve-fit of experimental data on similar material.

The results from the reproduced model were very similar to the experimental results, as can be seen in Figure 1, where the moment versus plastic rotation response of the experiment is compared against the monotonic simulation with excellent results. Additionally, the model showed significant yielding in the flanges and web within the reduced section coupled with buckling failure, which was consistent with the experimental results. An additional comparison was run between models with and without the explicit modeling of the erection bolts in the shear tab. Very minor differences were found, and in order to reduce computational time, the explicit modeling of the bolts was excluded in all subsequent models in this study.

Finite Element Models for the Study

Each of the models that were developed for this study followed the same approach described for the baseline model. The beam and the connection details were maintained identical to the baseline study configuration for all cases, and the parameters used for the study were the size of the column; whether a web doubler plate was provided; and the amount of axial force, varying in 20% increments from $0.2\phi P_n$ to $1.0\phi P_n$. Each specimen was modeled with five external restraints as shown in Figure 2. Lateral bracing was provided near the RBS section and at 84.0 in. from the beam connection [to replicate the experiment by Engelhardt (1998)] and at the beam compression flange outside of the RBS cut, which is an AISC 358-16 requirement for RBS connections. Columns are restrained from twisting freely by the presence of floor slabs and orthogonal floor beams, as well as by their own torsional stiffness; however, after an investigation of different torsional boundary conditions for the column, it was deemed most conservative to leave all torsional rotations free to take place. The column was perfectly pinned at the base, and a vertical roller was provided at the top to allow axial deformations of the column. The tip of the beam was restrained from displacing out-of-plane as well as from twisting.

The column sizes were picked to complement data in previous research and to ensure, whenever possible, the existence of strong column–weak beam behavior. Initial imperfections were introduced in all column sections based on the application of a linear combination of selected buckling modes for the column such that the fabrication tolerances for out-of-straightness would be upheld. For cases where a doubler plate was used, the panel zone was designed according to the AISC *Seismic Provisions* (2016b). All cases used 1.0-in.-thick continuity plates. Five axial loads levels were investigated: $0.2\phi P_n$, $0.4\phi P_n$, $0.6\phi P_n$, $0.8\phi P_n$ and $1.0\phi P_n$. Although the highest axial load level may seem impractical from a design perspective, it was used for comparative purposes with respect to load-displacement and column twist behavior. For axial loads of $0.2\phi P_n$ and $0.4\phi P_n$, all cases satisfied the strong column–weak beam criterion, while only the W14×426 and W30×191 passed for $0.6\phi P_n$. No case satisfied the strong column–weak beam criterion for axial loads of $0.8\phi P_n$ and higher. Table 2 shows which cases satisfied the strong column–weak beam criterion.

An important parameter in the evaluation of the response of connections to deep columns is the amount of twist that

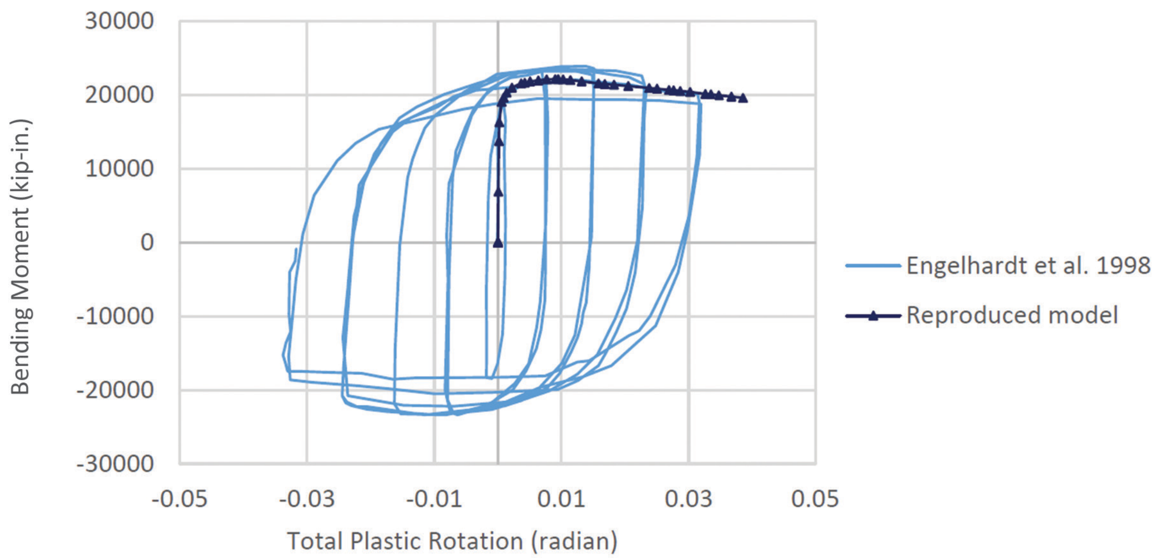

Fig. 1. Bending moment vs. total plastic rotation comparison.

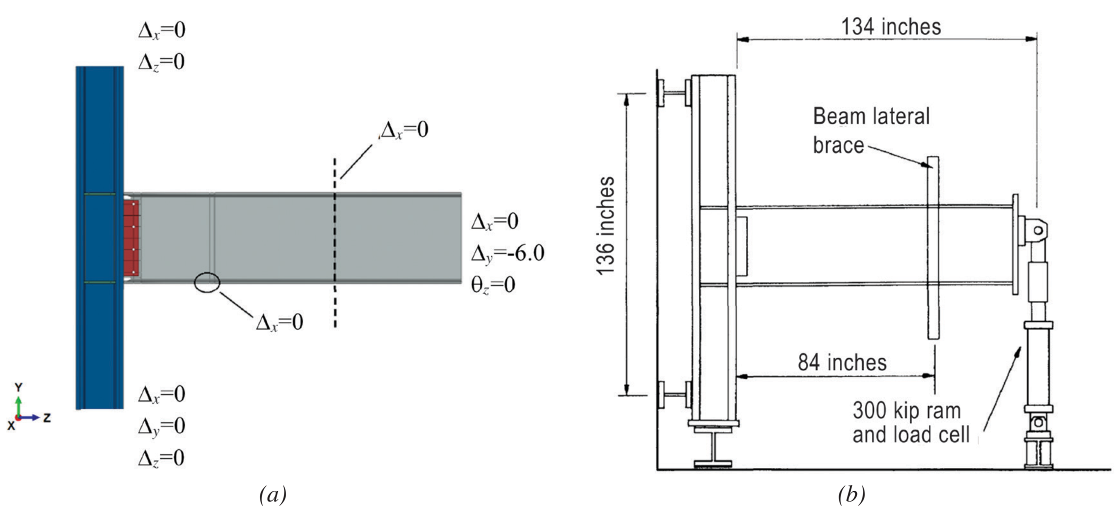

Fig. 2. (a) ABAQUS model with applied restraints and (b) test setup (Engelhardt et al.,1998).

Table 2. Strong Column–Weak Beam Criterion Satisfied				
Load	W14×426	W24×192	W27×194	W30×191
$0.2\phi P_n$	Yes	Yes	Yes	Yes
$0.4\phi P_n$	Yes	Yes	Yes	Yes
$0.6\phi P_n$	Yes	No	No	Yes
$0.8\phi P_n$	No	No	No	No
$1.0\phi P_n$	No	No	No	No

the columns undergo, potentially due to the lack of symmetry of the connection. The column twist was measured using the displacements at the tips of opposite column flanges as shown in Figure 3; the displacements were taken at the mid-height of the beam section, and their combination was divided by the diagonal distance between the points. This approach allowed for consideration of the movement of the whole section as opposed to only measuring the twist of the column flange connected to the beam relative to the column web, which resulted in column twist values that were negligible with respect to overall twisting rotations. In order to reduce the influence of the column size on this parameter, the twist angle was then scaled using the ratio of the flexural modulus to the torsional modulus. Scaling the column twist in such a fashion, while causing the parameter to lose its direct physical meaning, allowed a direct comparison of all cases investigated in this study, thus facilitating the process of drawing general conclusions.

In addition to the modeling of imperfections in the column, residual stresses were also considered in order to obtain a more realistic simulation of the column response. An equation for approximating the longitudinal residual stresses is presented in the AWS *Welding Handbook* (2001) and is reproduced here in Equation 1, where σ_x is the longitudinal residual stress in psi, σ_m is the maximum residual stress along the centerline of the weld (*k*-zone) in psi, y is the distance from the centerline of the weld (*k*-zone) in inches, and f is the half-width of the tensile residual stress zone, expressed in inches:

$$\sigma_x(y) = \sigma_m \left[1 - \left(\frac{y}{f}\right)^2\right]^{-\frac{1}{2}\left(\frac{y}{f}\right)^2} \tag{1}$$

Due to the similarity in distribution of residual stresses due to welding and those due to differential cooling/rolling processes, as well as the mathematical appeal of the AWS expression, this approach was used even for the case of a rolled section, likening the fillet weld size to the *k*-zone. Straight-line approximations of the residual stress were entered into the models by defining an initial stress condition for each element along the column length. When comparing this approach to residual stresses experimentally measured by Beedle and Tall (1960), this approach provides slightly higher residual stresses, which ensures a conservative approach.

The impact of residual stresses in the column sections was investigated in a sensitivity analysis that only included the W14×426 and W30×191 column sections subjected to $0.8\phi P_n$ axial load. Cases with and without a doubler plate were included for both column sections. Figure 4 shows the comparison of the load versus displacement graphs for the four cases studied. The graphs show a slight difference in initial stiffness. However, the cases with residual stresses

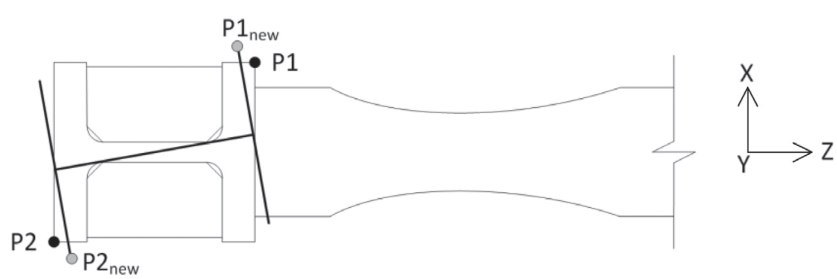

Fig. 3. Column twist.

Table 3. Peak Comparison for Residual Stress Cases				
Column	Residual Stress	Doubler Plate	Max Load (kips)	Beam Tip Displacement (in.)
W14×426	Excluded	No	161.6	2.12
		Yes	161.3	2.03
W14×426	Included	No	161.7	2.23
		Yes	161.1	2.20
W30×191	Excluded	No	160.8	2.39
		Yes	161.1	2.12
W30×191	Included	No	161.3	2.66
		Yes	160.9	2.11

reach a maximum load similar to the one reached in the cases without residual stresses. The peak capacity for all cases is reached within a range of 2.03- to 2.66-in. beam tip displacement as shown in Table 3. The only visible difference in the cases investigated is in the onset of yielding, which takes place sooner when residual stresses are considered but has little influence on the overall response of the connection. This conclusion is also supported by the results in Newell and Uang (2006), where it was shown that the effect of residual stresses on the high-axial load, high-drift demand behavior of columns is negligible.

Figure 5 shows the calculated twist versus story drift for the comparison of cases with residual stresses included and excluded. Note that the scale of the vertical axes for the graphs in Figure 5 is different: The W30 column twists considerably more than the W14; nevertheless, the general trend is noteworthy. For the cases with a web doubler plate, the scaled column twist appears largely unaffected by the inclusion of residual stresses as story drift begins to exceed approximately 2%. Cases without a doubler plate show a slightly more noticeable variation when residual stresses are included. Figure 5 shows that the scaled column twist appears to be smaller in cases that include residual stresses as the story drift begins to exceed approximately 2%.

Because these comparisons showed only minor differences in overall behavior, it was deemed appropriate to exclude residual stresses from the remainder of the study.

RESULTS

All cases with an applied axial load of less than $1.0\phi P_n$ reached similar maximum capacities of approximately 161 kips of applied transverse force and peaked at a similar beam tip displacement of about 2.0 in. as shown in Figure 6. This level of applied force is consistent with what caused the formation of a full plastic hinge in the reduced section in the control model. The only cases that differed were the ones with $1.0\phi P_n$ axial load for the three deep sections, which are represented by the six curves that unload prematurely in Figure 6. In these six cases, the columns underwent local inelastic buckling due to the combined action of shear from the connection and applied axial load before a plastic hinge could fully form in the beam. From the load versus displacement responses, no clear differences are visible among these cases regardless of panel zone strength. It was concluded that different levels of axial load do not affect the response significantly until very high axial loads are applied.

Figure 7 shows the scaled column twist versus the story drift for all cases studied, where the cases with a doubler plate have a solid marker. Three distinct groups of curves can be identified in that plot. The cluster of curves near the horizontal axis represent the response of the connections using the W14 column and show very little twist of the assemblies as drift increases. The intermediate group of curves represents the response of all connections to W24, W27 and W30 columns, in ascending order of sensitivity to twist, for all cases of axial load except the $1.0\phi P_n$ case. This shows that the scaled twist of the column sections increases with an increasing column depth. The six curves associated with large twists are the cases where column inelastic local buckling occurs before a full plastic hinge can form in the beam. The same chart also shows that the twist decreases with increasing axial loads until the design axial load is reached for the deeper column sections.

Furthermore, it was noted that cases with a web doubler plate undergo more twisting than the corresponding cases without a doubler plate. This is due to the combination of loss of symmetry induced by the doubler plate and a reduction in the concentration of plastic strains in cases with a doubler plate, as opposed to larger concentrations of plastic strains in weak panel zones, which result in a less severe local buckling in the RBS region and therefore a reduction in the torsional loading and the column twist. This phenomenon was also noted in Ricles et al. (2004), where the authors

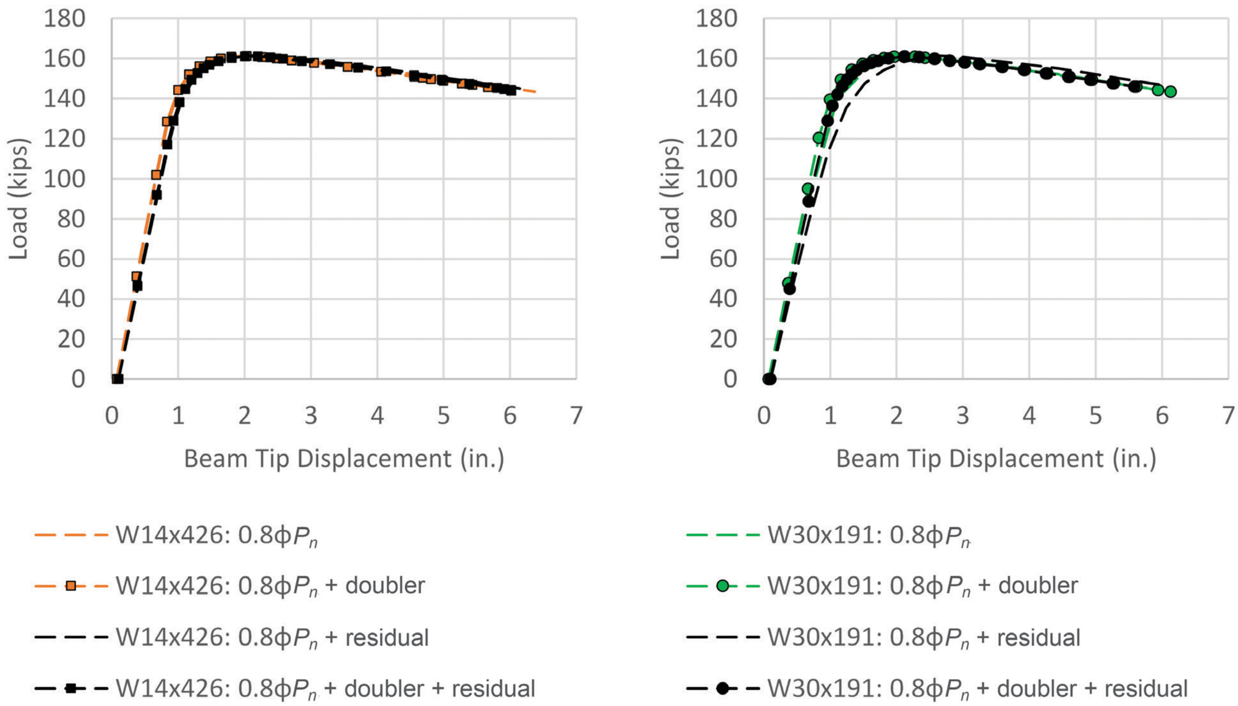

Fig. 4. Load vs. displacement curves for residual stress comparison.

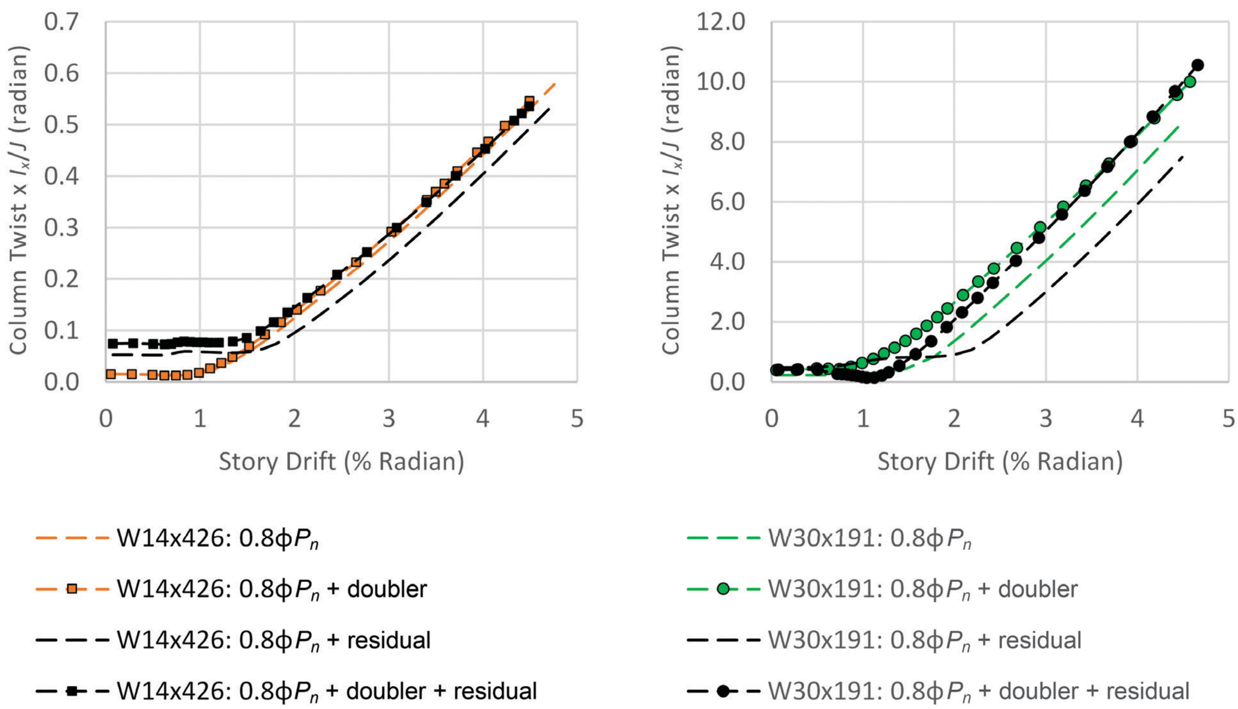

Fig. 5. Scaled twist vs. story drift curves for residual stress comparison.

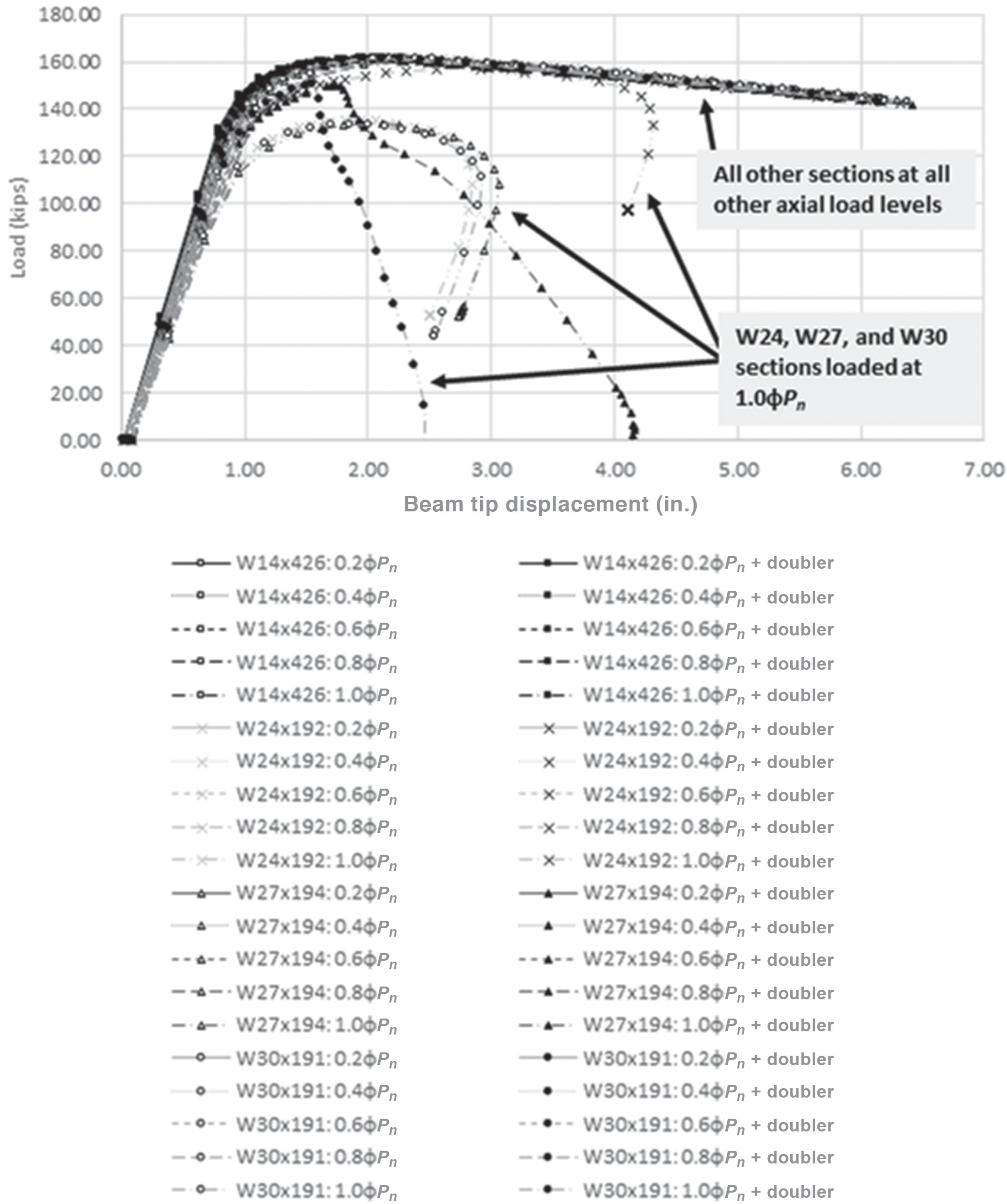

Fig. 6. Load vs. displacement curves for all cases.

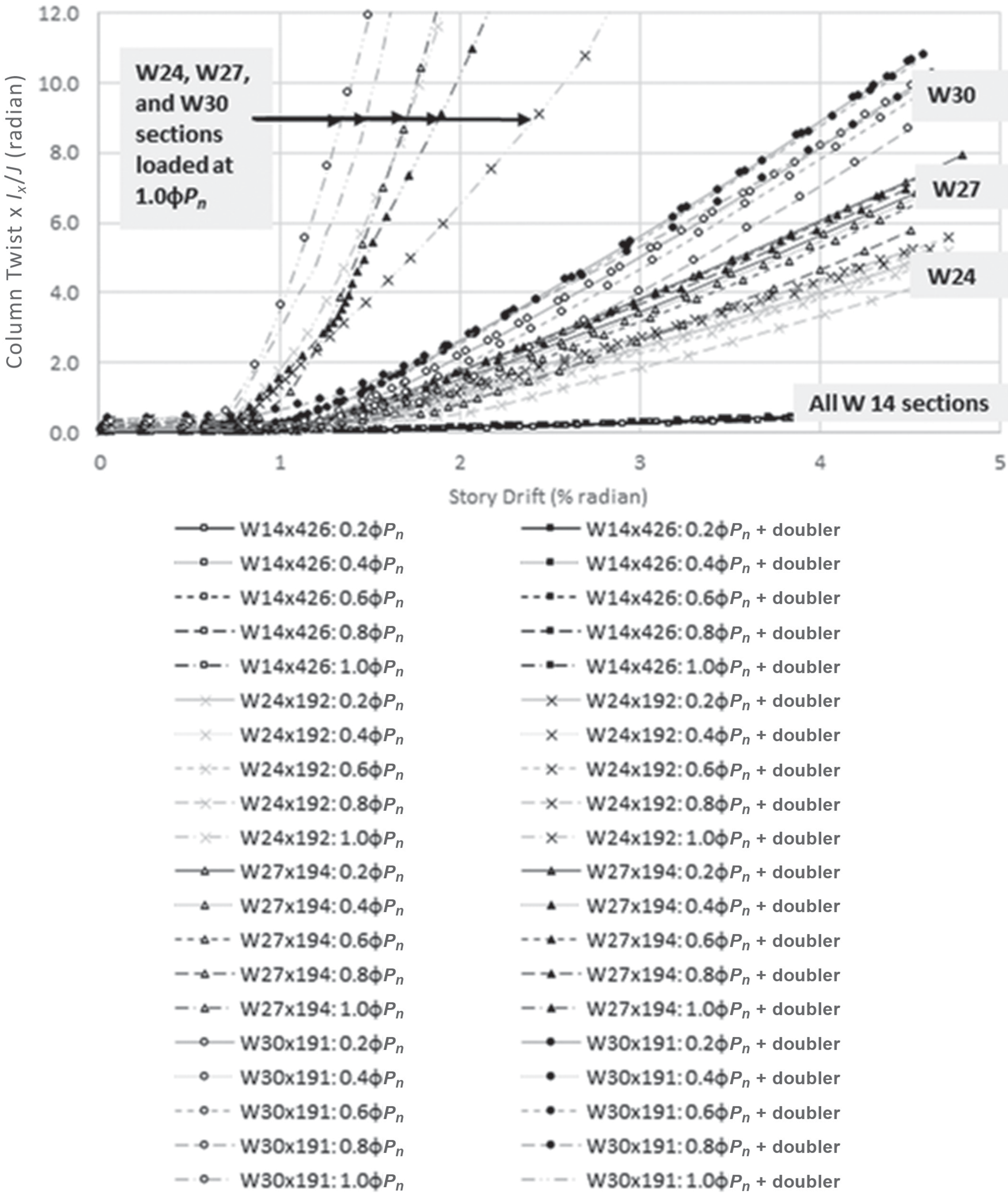

Fig. 7. Scaled twist vs. story drift curves for all cases.

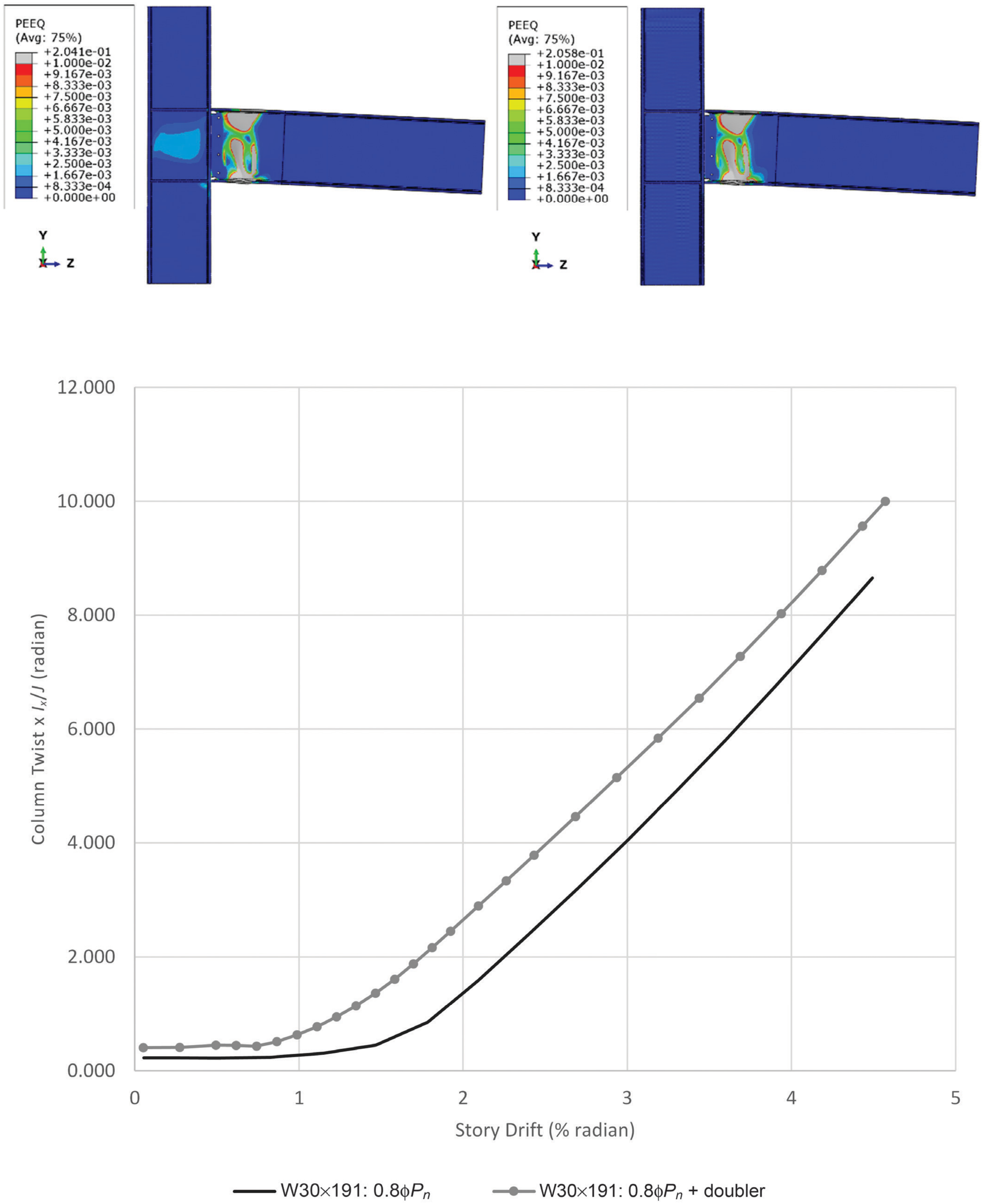

Fig. 8. W30×191 Assembly with 0.8ϕP_n axial load with PEEQ contours and drift vs. scaled twist. Unreinforced column web (top left) and with web doubler plate (top right).

also recommended against an unreinforced panel zone due to an increased potential for a ductile fracture of the connection. To further clarify this point, the difference in yielding demands in the panel zone for the case of a W30×191 column with $0.8\phi P_n$ axial load is shown in Figure 8, where the left assembly shows the unreinforced panel zone case and the one on the right is the specimen with a web doubler plate (which has been hidden for clarity). Figure 8 also shows the effects of the presence of the doubler plate on column twist for the same cases. When the doubler plate is present, the column begins twisting at relatively low drift demands, whereas yielding in the panel zone pushes the onset of twisting to higher demands.

CONCLUSIONS

Based on the results from the finite element models with varying column depths, varying levels of axial load, and medium and weak panel zones, it has been found that:

- Up to axial loads of $0.8\phi P_n$, the column depth does not appear to have any negative impact on the behavior of one-sided RBS connections. Even in the most conservative restraint configuration—without accounting for the presence of framing or of a floor slab—twisting of the column never rises to be a concern, allowing the connected beam to develop a plastic hinge in the RBS region as intended.
- When an axial force equal to the design axial load is applied to a deep column, the beam cannot fully develop its plastic strength as the column undergoes premature inelastic local buckling because of the combination of shear from the connection and applied axial force. Conversely, the W14 section investigated can withstand a force equal to the design axial load while concurrently resisting the shear force and bending moment combination required to develop a plastic hinge in the beam.
- Based on the results shown, it can be concluded that column twisting increases as the column depth increases. Column twisting decreases with increasing column axial loads, and columns reinforced with a doubler plate twist more than the corresponding cases with unreinforced webs. The cases with a reinforced web and low applied column axial forces exhibit the largest twist, while also undergoing the least amount of panel zone yielding. This shows an inverse relationship between panel zone yielding and amount of column twist. However, yielding in the column increases the risk of a ductile fracture in the column and threatens the strength of the connection.
- The presence of residual stresses does not appreciably change the behavior of the columns in the four cases studied under $0.8\phi P_n$ axial load.

This study shows that, especially when considering the restraint against twisting provided by the presence of orthogonal framing elements and floor systems, as well as of floor slabs, the use of deep columns in one-sided RBS moment connections does not lead to undesirable connection responses when column axial forces are below 80% of the column design axial capacity. Therefore, deep columns can be considered as a valid alternative to reduce the overall drift of a SMF, as long as proper detailing is provided to prevent hinging and local instability in columns as described in previous research (Newell and Uang, 2006).

The results presented warrant further analytical and experimental investigations in order to increase the reliability of RBS moment connections to deep columns, especially regarding the behavior of two-sided RBS connections, the effect of column twist on the fracture potential of the connection, the influence of column depth in weak-axis RBS connections, and the use of alternative column sections.

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