Seismic demands of steel moment resisting frames with inelastic beam-to-column web panel zones

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Abstract
This paper intends to provide quantitative seismic response characteristics of steel moment resisting frames (MRFs) with inelastic beam-to-column panel zone joints. Two modeling approaches are proposed for this purpose along with a methodology to consider the fracture potential at the bottom beam flange in steel MRFs due to panel zone kinking. Both approaches, which are validated with available experiments, preserve the behavioral insights of inelastic panel zones. Nonlinear static and dynamic analyses are then conducted to quantify the seismic demands of 32 prototype steel MRFs while their panel zones exhibit various levels of inelastic deformations. Engineering demand parameter hazard curves are developed to interpret the results for a risk-targeted seismic performance. The results demonstrate that steel MRFs with panel zone shear distortions of 10 to 15 times the shear distortion at yield, \( \gamma_y \), have a mean annual frequency of collapse ranging from \( 1.5 \times 10^{-4} \) to \( 2.5 \times 10^{-4} \), which is up to two times lower than corresponding results with code-compliant steel MRFs (i.e., \( \gamma \leq 4\gamma_y \)). It is shown that steel MRFs with inelastic deformations of \( 10\gamma_y \) in their panel zones, (a) enjoy up to 50% reduction in residual story drift ratios at a design basis earthquake; (b) their beam-to-column connections do not experience fractures due to panel zone kinking; and (c) local buckling in steel beams is very limited even at low probability of occurrence earthquakes. The above hold true when the current detailing and fabrication practice is employed. The findings have implications on seismic design and the post-seismic repairability of steel MRFs.

KEYWORDS
beam-to-column connection fractures, EDP hazard curves, inelastic panel zones, steel moment resisting frames, structural repairs

1 INTRODUCTION

In capacity-designed steel moment resisting frames (MRFs), the inelastic deformations in the beam-to-column web panel zones are usually limited.\(^1\)\(^2\) Instead, dissipative zones are situated near the beam ends and at the base of first story columns. While steel beams usually comprise seismically compact but slender cross sections near the current seismic compactness limits, local buckling is likely to occur even at modest lateral drift demands. Prior studies have shown that...
nonlinear geometric instabilities, such as local buckling, compromise a member’s flexural resistance and subsequently increase the collapse risk of steel MRFs during seismic events with a relatively low probability of occurrence.\textsuperscript{3,4} Although the annualized collapse risk of modern steel MRF buildings\textsuperscript{5} is within established limits of modern seismic standards,\textsuperscript{6} structural repairs may be deemed necessary depending on the extent of local buckling within a steel MRF’s dissipative zone. A potential way to limit structural damage near the beam ends is to allow for some participation of the beam-to-column web panel zone joints in the inelastic response of the respective beam-to-column connections. However, following the 1994 Northridge and 1995 Kobe earthquakes, it was found that steel MRF connections that featured inadequate base and weld material toughness requirements had a high fracture potential when their panel zones exhibited inelastic shear distortions higher than $4\gamma_y$\textsuperscript{7,8} (where $\gamma_y$ is the panel zone shear distortion at yield).

Recent test findings\textsuperscript{9–14} suggest that prequalified welded unreinforced flange-welded web (WUF-W) connections with highly inelastic panel zones (i.e., shear distortions up to $25\gamma_y$) are not susceptible to fractures at the beam flange-to-column flange region even at lateral drift demands of 4% rads. This is attributed to today’s improvements in weld specifications and fabrication practices.\textsuperscript{12,15–17} Noteworthy stating that for beam-to-column connections in multi-bay and space steel MRFs that feature beams with depths up to 500 mm, the fracture potential is usually less than that in connections with deeper beams.\textsuperscript{12,18,19}

Figure 1 shows the hysteretic response of pre-qualified WUF-W connections from recent experiments\textsuperscript{9} with limited ($2\gamma_y$) and high ($12\gamma_y$) inelastic panel zone shear distortion designs, $\gamma_d$. While both subassemblies featured the same cross sections, the fabrication cost in the second one was reduced because of the absence of doubler plates. Moreover, fabrication problems associated with welding near the k-area of steel columns are not a concern in this case. From the same figure, when inelastic deformations concentrate in the steel beams (see Figure 1A), cyclic deterioration in flexural strength is evident at lateral drift demands of 3% rads. Conversely, WUF-W connections that exhibit inelastic deformations in their panel zones (see Figure 1B) do not exhibit local instabilities prior to 5% story drift ratio (SDR). Prior work\textsuperscript{3,20,21} has shown that at these lateral deformation demands, the global stability of steel MRFs is controlled by P-Delta effects. Another aspect to be considered is that panel zone yielding within fully restrained beam-to-column connections relaxes the column twist demands due to the beam instability in its dissipative zone. This may be a concerning issue in designs that feature wide flange steel columns with deep cross sections (i.e., $d_b > 500$ mm).\textsuperscript{22}

A handful of studies highlighted that inelastic panel zones increase the lateral drift demands in steel MRFs by up to 30% and decrease their base shear capacity by 30%–25%.\textsuperscript{23–25} Conversely, research\textsuperscript{26,27} suggests that steel MRFs that exhibit some inelastic behavior in their panel zone joints enjoy a higher seismic collapse capacity than their elastic panel zone counterparts. Moreover, beam local buckling is delayed.\textsuperscript{28} However, the above studies utilized simplified modeling approaches and disregarded the likelihood of fracture within the beam-to-column connection attributable to panel zone kinking. Other studies investigated the influence of connection fractures on the seismic response of steel MRFs with pre-Northridge beam-to-column connections.\textsuperscript{29–32} However, these studies did not consider explicitly the effects of the panel zone inelastic deformation on the seismic response of the analyzed steel MRFs. To the best of our knowledge, there has not been a comprehensive study to benchmark the seismic demands of steel MRFs with inelastic panel zones from the onset of damage to structural collapse. On the other hand, there is a perception\textsuperscript{33,34} that steel MRFs exhibiting inelastic deformations in their panel zones are susceptible to collapse.

This paper proposes a new panel zone modeling approach that can be incorporated into system-level simulations without compromising important features of the panel zone and overall beam-to-column connection response. The seismic demands of archetype steel buildings with MRFs are then quantified by means of nonlinear static and dynamic analyses procedures. Several design parameters are interrogated, including the steel MRFs’ number of stories and bays as well
as the ‘allowable’ level of panel zone inelastic distortions. The collapse risk of the examined steel MRFs is quantified in the context of performance-based earthquake engineering. Hazard curves for global as well as local engineering demand parameters (EDPs) are developed to examine the influence of inelastic panel zone distortions on the seismic stability of steel MRFs at seismic intensities of interest to the engineering profession.

2 | PROPOSED MODELING APPROACH FOR BEAM-TO-COLUMN CONNECTIONS WITH INELASTIC PANEL ZONES

2.1 | Behavioral insights

Seminal studies\textsuperscript{35} at the University of California at Berkeley in the 1970s demonstrated that beam-to-column web panel zone joints experience shear yielding, which is a stable damage mechanism, under cyclic loading. Referring to Figure 2A, the kinematics of the panel zone joint essentially imply that the top and bottom edges of the panel zone remain plain under lateral loading, whereas the column web and flanges exhibit shear and flexural modes of deformation, respectively, depending on the panel zone geometry. Moreover, the evolution of shear yielding, the cyclic hardening, and the axial-to-shear load interaction are important considerations in the overall hysteretic behavior of panel zones. Two modeling approaches are proposed in the subsequent sections to preserve some of the above characteristics within a frame analysis nonlinear finite element program. Both approaches are made publicly available (https://github.com/RESSLab-Team).
2.2 Shell element modeling approach

Figure 2B depicts schematically the proposed modeling approach for a typical panel zone geometry that is comprised of a steel beam and column with depths \(d_b\) and \(d_c\), respectively. Plane stress conditions are assumed for the column web and flanges. These are modeled with shell elements with a thickness equal to the column web, \(t_{cw}\), and the column flange width, \(b_{cf}\), respectively. A high fidelity four-node shell element is first developed and implemented in the Open System for Earthquake Engineering Simulation (OpenSees)\textsuperscript{36} platform for this purpose. The developed element (so-called FourNodeQuad2D3DOF), which is suitable for two dimensional (2D) nonlinear analyses, considers both the translational and rotational degrees-of-freedom (DOFs). The proposed element employs a combined isotropic and kinematic hardening constitutive law within the framework of J2 plasticity.\textsuperscript{37} In order to satisfy the panel zone kinematics, rigid links are employed between the column top and bottom nodes and the panel zone top and bottom edge nodes, respectively (see Figure 2B). These links constrain the translational and the rotational DOFs for this purpose. The proposed modeling approach captures the panel zone bending and shear deformation modes. This is achieved by connecting the beam ends with a pin directly to the column’s rigid link corner nodes, without imposing any constraints to the shell element nodes of the panel zone. The beam moment is transferred as a force couple, \(V_{pz}\), through beam rigid links as shown in Figure 2B. The translational and rotational DOFs are constrained for the rigid links between the beam end nodes and the bottom panel zone corners. Between the top panel zone corners and the beam end nodes, the translational DOF in the \(y\) direction is released so as the axial force passes through the panel zone shell elements. Therefore, the axial-to-shear load interaction in the panel zone is explicitly considered.

The proposed modeling approach is facilitated through an automatic mesh generator. This considers the input panel zone geometry and the desired elements per web and flange thickness. Based on a sensitivity analysis that was conducted as part of the present paper, it was found that one element per flange thickness and eight elements per web thickness provide superior results.

2.3 Macro-model approach

In this case, the proposed modeling approach is motivated by the parallelogram model, which was originally proposed by Gupta and Krawinkler.\textsuperscript{38} This model is illustrated in Figure 2C. The proposed modeling approach retains simplicity and computational efficiency within a frame analysis nonlinear finite element program. Rigid links are assumed for the panel zone edges and pinned connections at three of the panel zone corners. The hysteretic behavior of the panel zone is preserved in a rotational spring, which is located at one of the panel zone corners as shown in Figure 2C. This spring is assigned a trilinear panel zone moment, \(M_{pz}\), versus panel zone shear distortion, \(\gamma\), relationship. While the parallelogram model does not account explicitly for the bending deformation mode of a panel zone, this is implicitly considered based on the recently proposed model by Skiadopoulos et al.\textsuperscript{39} In particular, the panel zone elastic stiffness, \(K_e\), is computed based on Equations (1)–(3),

\[
K_e = \frac{M_{pz}}{\gamma} = \frac{K_s \cdot K_b}{K_s + K_b} \tag{1}
\]

\[
K_s = \frac{A_v \cdot G}{d_b - t_{bf}} = \frac{t_{pz} \cdot (d_c - t_{cf}) \cdot G}{d_b - t_{bf}} \tag{2}
\]

\[
K_b = \frac{12 \cdot E \cdot I_c}{d_b - t_{bf}} \tag{3}
\]

where \(K_s\) and \(K_b\) are the elastic stiffnesses corresponding to the shear and bending mode of deformation of the panel zone, respectively; \(A_v\) is the panel zone shear area; \(G\) is the shear modulus of the steel material; \(t_{pz}\) is the panel zone thickness, including the column web and doubler plate(s) thicknesses, if any; \(d_c\) is the depth of the column cross section; \(t_{cf}\) is the thickness of the column flange; \(E\) is the Young’s modulus of the steel material; \(I_c\) is the second moment of area of the column cross section (including the doubler plate thickness, if any) with respect to the strong axis of the column’s cross section.
Referring to Figure 2C, the moment resistance of the panel zone is defined at \( \gamma_y \), \( 4\gamma_y \) and \( 6\gamma_y \), according to Equation (4),

\[
M_{pz} = \frac{f_y}{\sqrt{3}} \cdot \left[ a_{weff} \cdot (d_c - t_{cf}) \cdot t_{cw} + a_{fkeff} \cdot (b_{cf} - t_{cw}) \cdot 2t_{cf} \right] \cdot (d_b - t_{bf})
\]

where \( f_y \) is the steel material yield stress; \( a_{weff} \) and \( a_{fkeff} \) are shear stress coefficients for the web and the flanges, respectively, that depend on the level of inelastic shear distortion within the panel zone. These coefficients can be estimated according to Skiadopoulos et al. This model depicts well the contribution of the column flanges to the overall shear resistance of the panel zone, especially in designs that feature stocky column cross sections. It should be noted that, unlike the first approach, the macro-model discussed herein does not capture the cyclic hardening and shear-to-axial load interaction within the panel zone.

2.4 Validation and comparison with system level behavior

Figure 2D shows indicative comparisons between the simulated and measured panel zone moment versus shear distortion relationships from prior experimental work. The panel zone shear distortion reached up to \( 12\gamma_y \) in this case. The proposed shell element model provides a remarkable accuracy in predicting the panel zone behavior, including the onset of shear yielding and the cyclic hardening. The simulated response based on the macro-model is a modest representation of the experimental data. Same observations hold true for validations with a rich set of collected experimental data, which are not shown here due to brevity.

To quantify the impact of the different panel zone modeling approaches on the structural response of steel MRF buildings, a four-story, three-bay steel MRF is analyzed with both modeling approaches. The evaluation was conducted with nonlinear response history analysis based on the set of 44 far-field ground motions from FEMA P695. The plan view and the modeling assumptions of prior studies are employed for WUF-W connections. In this case, \( \gamma_d = 10\gamma_y \) is considered. Figure 3 depicts the median values of the peak \( SDR \) and of the peak normalized panel zone distortions, \( \gamma/\gamma_y \) along the steel MRF height. The 16th and 84th percentiles are also superimposed in the same graph. Both panel zone modeling approaches lead to nearly identical distributions for the above mentioned EDPs of interest. Similar observations hold true for the beam and column responses, the absolute floor accelerations and the residual \( SDR \) distributions along the steel MRF height. Therefore, to expedite the nonlinear analyses, which are discussed in the subsequent sections, the macro-model approach is adopted.

2.5 Modeling of beam flange fracture

In this section, a methodology is formulated to consider the fracture potential in WUF-W beam-to-column connections with highly inelastic panel zone joints. The model updating technique of OpenSees is utilized for this purpose. Figure 4 shows a schematic illustration of the adopted workflow. In brief, at each analysis time step, \( k \), the \( SDR_{ik} \), at floor \( i \) and
the normalized shear distortion, $y_{ij}/y_{ij}$, for each panel zone $ij$ are computed. For the given $SDR_{ij}$ and $y_{ij}/y_{ij}$, the probability of fracture $P_{ij}$ is computed for adjoining steel beams with their bottom flange in tension (denoted as Beam $ij$ in Figure 4). It is assumed that beam fracture is evident at this location, because the top flange is considered to be restrained by the concrete slab of the floor system. The computed $P_{ij}$ is based on bivariate log-normal distributions for WUF-W post-Northridge connections\(^{40}\) with input predictor variables the $SDR_{ij}$ and $y_{ij}/y_{ij}$ as shown in Figure 4. To claim fracture in Beam $ij$ at time $k$, the $P_{ij}$ should be greater or equal to a targeted threshold, $P_{max}$. In this case, model updating is applied at the rotational zero-length element that represents the response at the Beam $ij$ end. The flexural resistance of the steel beam is set equal to zero at this step, $k$, as shown in Figure 4. This is consistent with prior experimental work.\(^{9,10,12,13}\)

A sensitivity analysis was conducted to determine the influence of the assumed $P_{max}$ value to the steel MRF’s seismic response. Three threshold values were explored, for $P_{max} = [25, 50, 100]$%. For all EDPs of interest, at DBE, the simulations confirmed that there were no connection fractures regardless of the $P_{max}$ value. At MCE, only the $P_{max} = 25\%$ showed beam end fractures in the steel MRFs with highly inelastic panel zones (i.e., $y_{d} = 15y_{y}$). However, these fractures do not practically affect any of the local and global EDPs of interest. In the subsequent nonlinear building simulations, $P_{max} = 50\%$ is conservatively assumed.

### 3 ARCHETYPE STEEL BUILDINGS

Thirty-two archetype steel office buildings are designed according to.\(^{1,6,17,41}\) The design location is assumed to be in urban California (coordinates: 33.966°N, −118.162°W). A soil class D and risk category II are assumed as per ASCE/SEI 7–16.\(^{6}\) The steel buildings consist of two perimeter steel MRFs, two orthogonal concentrically braced frames (CBFs), and a gravity frame system as shown in Figure 5A. Four-, eight-, 12- and 20-story steel buildings are considered. The steel MRFs in the East-West loading direction have either three or five bays as shown in Figure 5B and are designed as steel special moment frames. The typical story height is 4 m in all examined cases. The first story height is 4.3 m for the four-story steel MRF. The rest of the steel MRFs have a first story height of 4.2 m. Beams and columns are made of A992, Gr. 50 (i.e., nominal yield stress, $f_{y} = 345$ MPa) with a 100mm thick slab that rests on a 90mm thick steel deck. Pre-qualified WUF-W beam-to-column connections\(^{17}\) are considered. Four steel MRF designs are conducted. The beam-to-column web panel zones are designed with the following targeted panel zone distortions, $y_{d} = [1, 4, 10, 15]y_{y}$. The first two values comply with AISC 360-16,\(^{4}\) whereas the last two design variations do not. The panel zone shear resistance, $R_{y}$,\(^{4}\) over the panel zone shear demand, $R_{d}$,\(^{41}\) values are reported in Table 1 for all the designs. Column splices are positioned at the mid-height of every other story.

The archetype steel buildings are designed based on response spectrum analysis. The first mode periods, $T_{1}$, of the buildings in the East-West direction are also summarized in Table 1 for comparison purposes. For the same structural height, the first mode period is fairly similar between buildings regardless of the targeted panel zone strength and the number of bays. This is attributed to the fact that the steel MRFs are drift-controlled. Referring to the same table, the range of the strong-column/weak-beam (SCWB) ratios is fairly similar between designs. The panel zone contribution to the MRF lateral deformations is considered as part of the design process. In steel MRF designs where panel zones were designed to attain 10 or 15$y_{y}$, doubler plates were either not deemed to be necessary or they featured thicknesses up to
Three-and-five-bay, four-story steel buildings. (A) Typical plan views. (B) Elevation views of the analyzed MRFs

The above reduce the anticipated fabrication costs due to weld savings. A summary of the complete designs of the 32 MRFs presented herein are made publicly available from https://www.doi.org/10.5281/zenodo.5962407.

4 | NONLINEAR BUILDING MODELS

Two-dimensional models of the steel buildings are developed in OpenSees.\textsuperscript{36} The steel members of the perimeter steel MRFs shown in Figure 5B are idealized with elastic beam-column elements and concentrated plasticity elements at their ends. The modified Ibarra-Medina-Krawinkler deterioration model\textsuperscript{14} is considered to model cyclic deterioration in flexural strength and stiffness of the respective steel beams and columns. For this purpose, the modeling recommendations by Lignos and Krawinkler\textsuperscript{42} and Lignos et al.\textsuperscript{43} are employed, respectively. The panel zones are modeled as explained in Section 2 based on the macro-model approach.

The nonlinear geometric effects are explicitly considered during the nonlinear analyses of the considered models with the P-Delta geometric transformation. The destabilizing effects of the gravity load are considered through a leaning column, which is connected to the steel MRF through axially rigid links. The axial load assigned to the leaning column equals half the gravity load, excluding that assigned directly to the steel MRF columns based on their tributary area. Two percent damping ratio is assigned at the first and third modes of the steel MRFs. Viscous damping is considered with the Rayleigh model based on the procedures discussed in Zareian and Medina.\textsuperscript{44} While prior studies\textsuperscript{5,38,45} have emphasized on the beneficial effects of the gravity framing system on the seismic stability of steel frame buildings, herein, the lateral stiffness and strength of the gravity framing system is, conservatively, neglected. While the prototype frames in this study are designed as non-composite, the role of the composite slab may be explicitly considered in the frame analysis models according to prior related studies.\textsuperscript{2,46}

5 | NONLINEAR STATIC ANALYSIS

Nonlinear static analysis (referred to as pushover analysis hereinafter) based on a first mode lateral load pattern of each steel MRF is conducted to quantify the effect of the targeted panel zone distortions, $\gamma_d$, on the static overstrength factor,
### Archetype steel MRF first mode periods and global performance factors

| Floor / Bay | $\gamma$ | $R_u / R_a$ | $T_1$ (s) | SCWB ratio | $\Omega$ | $\mu_T$ | $\mu_{Sa,C}$ | $\sigma_{In,Sa,C}$ |
|-------------|----------|------------|--------|-----------|---------|---------|-------------|-----------------|
| 4-story 3-bay | $\gamma_y$ | 1.25 | 1.28 | 1.4–2.2 | 2.30 | 3.98 | 1.65 | 0.42 |
| | $4\gamma_y$ | 1.03 | 1.29 | | 2.28 | 4.21 | 1.70 | 0.42 |
| | $10\gamma_y$ | 0.82 | 1.31 | | 2.21 | 4.85 | 1.67 | 0.39 |
| | $15\gamma_y$ | 0.79 | 1.32 | | 2.17 | 5.23 | 1.78 | 0.37 |
| 5-bay | 1.26 | 1.28 | 1.4–2.0 | 2.60 | 3.70 | 1.55 | 0.37 |
| | $4\gamma_y$ | 1.04 | 1.29 | | 2.58 | 3.89 | 1.64 | 0.41 |
| | $10\gamma_y$ | 0.85 | 1.31 | | 2.52 | 4.34 | 1.74 | 0.36 |
| | $15\gamma_y$ | 0.75 | 1.32 | | 2.45 | 4.86 | 1.89 | 0.35 |
| 8-story 3-bay | 1.22 | 2.04 | 1.2–2.3 | 2.56 | 3.35 | 0.96 | 0.36 |
| | $4\gamma_y$ | 1.00 | 2.08 | | 2.52 | 3.57 | 0.99 | 0.35 |
| | $10\gamma_y$ | 0.85 | 2.11 | | 2.45 | 4.08 | 1.04 | 0.34 |
| | $15\gamma_y$ | 0.79 | 2.13 | | 2.39 | 4.50 | 1.09 | 0.33 |
| 5-bay | 1.23 | 2.06 | 1.1–2.2 | 2.77 | 3.03 | 0.96 | 0.36 |
| | $4\gamma_y$ | 0.98 | 2.09 | | 2.73 | 3.18 | 0.99 | 0.35 |
| | $10\gamma_y$ | 0.84 | 2.12 | | 2.65 | 3.53 | 1.04 | 0.34 |
| | $15\gamma_y$ | 0.74 | 2.14 | | 2.59 | 3.83 | 1.09 | 0.33 |
| 12-story 3-bay | 1.23 | 2.41 | 1.1–2.1 | 3.46 | 2.92 | 0.93 | 0.36 |
| | $4\gamma_y$ | 0.97 | 2.45 | | 3.41 | 3.06 | 0.91 | 0.35 |
| | $10\gamma_y$ | 0.81 | 2.49 | | 3.34 | 3.35 | 1.00 | 0.35 |
| | $15\gamma_y$ | 0.75 | 2.50 | | 3.27 | 3.64 | 1.04 | 0.34 |
| 5-bay | 1.21 | 2.47 | 1.1–2.7 | 3.68 | 2.69 | 0.92 | 0.37 |
| | $4\gamma_y$ | 0.99 | 2.51 | | 3.61 | 2.90 | 0.91 | 0.35 |
| | $10\gamma_y$ | 0.84 | 2.55 | | 3.48 | 3.45 | 0.99 | 0.35 |
| | $15\gamma_y$ | 0.76 | 2.57 | | 3.37 | 3.87 | 1.03 | 0.34 |
| 20-story 3-bay | 1.24 | 3.91 | 1.1–2.1 | 2.98 | 1.57 | 0.50 | 0.32 |
| | $4\gamma_y$ | 0.99 | 3.95 | | 2.94 | 1.63 | 0.52 | 0.32 |
| | $10\gamma_y$ | 0.83 | 3.98 | | 2.89 | 1.73 | 0.52 | 0.31 |
| | $15\gamma_y$ | 0.74 | 4.00 | | 2.81 | 1.91 | 0.51 | 0.31 |
| 5-bay | 1.23 | 3.86 | 1.1–2.2 | 3.16 | 1.65 | 0.50 | 0.32 |
| | $4\gamma_y$ | 0.97 | 3.91 | | 3.09 | 1.75 | 0.52 | 0.32 |
| | $10\gamma_y$ | 0.85 | 3.94 | | 3.01 | 1.91 | 0.52 | 0.31 |
| | $15\gamma_y$ | 0.75 | 3.97 | | 2.92 | 2.14 | 0.51 | 0.31 |

$\Omega$, and the period-based ductility factor, $\mu_T$. These parameters, which are shown in Figure 6A for the four-story, three-bay steel MRFs, are defined according to FEMA P-695. In this figure, the base shear, $V$, is normalized with respect to the seismic weight, $W$; the roof drift ratio, $\delta_r / H$, is computed based on the roof displacement, $\delta_r$, over the total height, $H$, of the steel building. Superimposed in the same figure is the normalized design base shear, $V_d / W$, which is identical in all design cases shown in the figure. The results suggest that the elastic stiffness and the base shear capacity (noted as $V_{max}$ in Figure 6A) are nearly the same in all cases.

Referring to Figure 6A, the static overstrength, $\Omega$, and the period-based ductility, $\mu_T$, are summarized in Table 1 for all the examined MRFs. The variation of both parameters between the three- and the five-bay MRFs is insignificant. Except for a few MRF designs, $\Omega$ appears to be lower than the minimum overstrength factor for special moment frames, that is, $\Omega_d = 3$, as specified in ASCE/SEI 7–16. However, this is because the stabilizing effects of the gravity framing system have been neglected. Interestingly, Table 1 suggests that $\Omega$ is practically not influenced by the targeted panel zone shear distortion designs. The $\mu_T$ generally decreases while the structural height increases. In taller MRFs, their primary collapse mechanism involves lesser stories due to the shear mode of deformation. Moreover, the $\mu_T$ of steel MRFs increases by 20%
Figure 6B shows the normalized floor displacements along the building heights for the 4-story steel MRF designs. The floor displacements, $\delta_i$, are normalized with respect to the building heights, $H$. The results suggest that the collapse mechanisms of the employed designs are not sensitive to the targeted panel zone shear distortions even at roof drift ratios of 6% rads. Figure 6C depicts the maximum (among all bays) plastic rotations, $\theta_{pl}$, of the first story columns at the base and at the floor beams along the building height at a roof drift ratio of 3% rads. In the same figure, we have superimposed the precapping plastic rotations, $\theta_p$, of the same members according to. These values indicate the onset of local buckling within the dissipative zone of the respective member(s). Interestingly, in steel MRF designs with $\gamma_d \geq 10\gamma_y$, although flexural yielding occurs in the steel beam ends, these do not experience local buckling. Conversely, designs with $\gamma_d \leq 4\gamma_y$ experience local buckling within the dissipative zones of steel beams, particularly in the first two stories of the steel MRF. This comparison demonstrates the value of balancing the panel zone strength with respect to that of the adjoining steel beam(s).

Figure 6D depicts the normalized panel zone shear distortions, $\gamma/\gamma_y$, along the building height at a targeted roof drift ratio of 3% rads, that is, characteristic of an MCE with a 2% probability of exceedance over a 50-year building life expectancy. It is observed that for the designs with $\gamma_d \leq 4\gamma_y$, the panel zones reach the targeted distortion, since beams reach their capping moment at the targeted roof drift ratio. Conversely, in designs where $\gamma_d \geq 10\gamma_y$, the targeted inelastic shear distortions are not attained. This explains the increased $\mu_T$ values in the latter case. Same findings hold true for the rest of the designs but the results are not shown herein due to brevity.

Observations from nonlinear static analyses suggest that when inelastic deformations are more balanced between the steel beams and the beam-to-column web panel zone, local buckling is not pronounced even at lateral drifts associated with an MCE, thereby minimizing the need for structural repairs in the aftermath of earthquakes. Due to well-known limitations of pushover analysis, nonlinear response history analyses are carried out in the subsequent section for quantifying both local and global EDPs of interest for all the examined steel MRFs.
6 | NONLINEAR RESPONSE HISTORY ANALYSIS

6.1 | Collapse risk evaluation

This section summarizes the effect of the targeted panel zone inelastic shear distortions on the collapse risk of the examined steel MRFs. Incremental dynamic analysis (IDA) is conducted for the set of 44 far-field ground motions of FEMA-P695. For a given ground motion, its seismic intensity is scaled till a single story or a number of stories displaces laterally by more than 15% rads and the corresponding story shear resistance becomes zero. This definition of sidesway collapse is consistent with measurements from shake table collapse experiments on steel MRFs. The adopted intensity measure (IM) for the ground motion scaling is the first mode, 5% damped spectral acceleration, $S_a(T_1, 5\%)$. Although more efficient and sufficient IMs exist, at this time, the lack of available seismic hazard curves as a function of these IMs hinders their adaptation in the present study. The seismic hazard curves of the 4-, 8-, 12-, and 20-story, three-bay steel MRFs at the design location are shown in Figure 7. These are adopted from the United States Geological Survey (USGS) online tool.

Figure 8A shows representative IDA curves for the three-bay, four-story steel MRF with $\gamma_d = 10\gamma_y$. The median, the 16th and 84th percentiles of the IDA curves are superimposed for reference based on counted statistics. The peak SDR at which structural collapse occurs ranges from 6% to 11% rads. Figure 8B shows the IDA curves with respect to the peak $\gamma/\gamma_y$ along the height of the steel MRF. The results suggest that when $S_a(T_1, 5\%)$ is larger than 1.2 g (i.e., at MCE), the peak panel zone inelastic shear distortions are capped at about $10\gamma_y$. This is because the adjacent steel beams attain their capping moment at the same lateral drift demands, as intended. From the same figure, the peak $\gamma$ ranges between $8\gamma_y$ and $14\gamma_y$. 
Figures 9A and 9B compare the fitted log-normal collapse fragility curves of the $\gamma_d = \gamma_y$ and $\gamma_d = 10\gamma_y$ design cases for the 4-story, three- and five-bay steel MRFs. In the collapse fragility computations, spectral shape effects that were disregarded from the ground motion selection are considered through an adjustment of epsilon as per Haselton et al.\textsuperscript{52} Noteworthy stating that the median collapse intensity of all steel MRFs with inelastic panel zone designs is almost 10%–20% higher than that of their elastic panel zone design counterparts. Because all designs are drift-controlled, the collapse fragility curves are not practically affected by the number of the steel MRF bays. Similar trends hold true for the rest of the MRF designs. This can also be seen in Table 1 from the tabulated median, $\mu_{S\alpha,C}$ and the logarithmic standard deviation, $\sigma_{lnS\alpha,C}$, of the collapse intensities of the respective steel MRFs. Interestingly, the $\sigma_{lnS\alpha,C}$ values for the examined steel MRFs are not practically influenced by the extent of the panel zone inelastic shear distortion. The seismic collapse risk of the steel MRFs is evaluated by computing the mean annual frequency of collapse, $\lambda_c$. To compute $\lambda_c$, the collapse fragility curves of the steel MRFs are numerically integrated over the seismic hazard curve that corresponds to the design location,\textsuperscript{53} as per Equation (5). For a given intensity, $im$, the probability of collapse, $P(C|im)$, is computed based on the empirical cumulative probability distribution. The term $d\lambda(im)/d(im)$ is then computed from the slope of the hazard curve at the given $im$, while $d(im)$ is the increment over which the numerical integration is applied as per Eads et al.\textsuperscript{54} By assuming that earthquakes follow a Poisson distribution, the probability of collapse over 50 years, $P_c (50 \text{ years})$ is then computed in Figures 9C and 9D for the three- and five-bay steel MRF, respectively.

$$\lambda_c = \int_{0}^{\infty} P(C|im) \cdot \frac{d\lambda(im)}{d(im)} \cdot d(im)$$  \hspace{1cm} (5)

The results suggest that the earthquake-induced collapse risk is practically not influenced by the number of the steel MRF bays. This is attributed to the fact that the steel MRF designs are drift-controlled as discussed earlier; hence P-Delta effects dominate their seismic stability.\textsuperscript{21,55} The collapse risk tends to decrease while the number of stories increases because the seismic hazard is typically lower for long-period structures (see Figure 7). The 1% limit of the probability of
collapse over 50 years of ASCE/SEI 7–16 is superimposed for reference in Figures 9C and 9D. While steel MRFs with elastic panel zone designs do not respect the above limit, it should be stressed that the stabilizing effects of the gravity framing have been neglected in the collapse simulations. Interestingly, steel MRFs with panel zones attaining $10 - 15\gamma_y$ have smaller or equal collapse risk with code-compliant steel MRFs. For instance, for the four-story, three-bay steel MRF, the collapse risk decreases by more than 30% when panel zones are designed to attain $15\gamma_y$ contrary to the design with $\gamma_d = \gamma_y$. Moreover, the findings suggest that steel MRFs exhibiting inelastic panel zone deformations are not necessarily prone to soft story collapse mechanisms. This is further substantiated in Figure 10 that shows the median peak SDRs along the steel MRF heights at the last stable point of each IDA curve for the three-bay steel MRFs. The peak SDR distributions along the building heights are nearly the same regardless of the targeted panel zone inelastic shear distortion.

6.2 Seismic demands at discrete seismic intensities of interest

This section quantifies the seismic demands of the examined steel MRFs at two seismic intensities of interest, namely DBE and MCE. The focus is on global EDPs, such as the peak SDRs and the residual SDRs along the steel MRF heights. Moreover, of interest are local EDPs, such as the peak panel zone inelastic shear distortions, the peak plastic rotations of the steel beams as well as the number of potential beam bottom flange fractures. Among other reasons, the above EDPs are strongly related to structural repairs in steel MRFs in the aftermath of earthquakes.

Figure 11 shows comparisons of the EDPs of interest between the $\gamma_d = \gamma_y$ and the $\gamma_d = 10\gamma_y$ cases. The median and the 84th percentiles of the four-story, three-bay MRFs are shown for reference. The peak SDR demands are somewhat reduced in cases where steel MRFs are designed with highly inelastic panel zones (i.e., $\gamma_d = 10\gamma_y$) relative to their elastic panel zone counterparts. These reductions are more pronounced at MCE (see Figures 11A and 11E).

Figures 11B and 11F suggest that the residual SDRs are reduced by 50%–100% in steel MRFs with inelastic panel zones compared to those in their elastic panel zone counterparts. Prior studies have highlighted the impact of residual SDR on economic losses of MRF buildings due to demolition. Noteworthy stating that at DBE, steel MRF designs with $\gamma_d = 10\gamma_y$, do not exhibit residual SDR values of more than 0.4% rads, on average, contrary to 0.6% rads for the designs with $\gamma_d = \gamma_y$. At MCE, the same designs attain, on average, about 0.8% and 1.2% residual SDRs, respectively. Interestingly, the 84th percentile of the analyzed cases at MCE suggests that the residual SDR is 2% or less along the steel MRF height when panel zones are designed to exhibit inelastic shear distortions of $10\gamma_y$. Conversely, steel MRFs with elastic panel zones are likely to experience a residual SDR in the order of 4% at an MCE. While in both cases building demolition is likely, in the latter the likelihood of structural collapse is high in a typical mainshock-aftershock seismic event series. The general consensus from the above findings is that leveraging the beneficial stable hysteretic behavior of beam-to-column web panel zones greatly reduces the anticipated residual SDRs in steel MRFs after a low probability-of-occurrence earthquake. This is further substantiated from the local EDP responses of the examined steel MRFs.

Figures 11C and 11G suggest that in steel MRFs with $\gamma_d = 10\gamma_y$, the panel zones contribute 5 to 15 times more to the SDR for the DBE and the MCE, respectively, compared to steel MRFs with elastic panel zones. Steel MRF panel zones designed with $\gamma_d = 10 - 15\gamma_y$ do not experience, on average, more than 6 – 9\gamma_y, respectively. This suggests that fracture due to kinking is highly unlikely, given that the fabrication of beam-to-column connections follows the current practice.
FIGURE 11  Effect of targeted panel zone distortion on engineering demand parameters of interest for the four-story, three-bay MRF (top Figures A to D refer to DBE and bottom Figures E to H refer to MCE)

FIGURE 12  Exceedance functions for the number of beam end fractures per MRF at MCE. A, Four-story, three-bay MRFs with varying $\gamma_d$. B, Three-bay MRFs with $\gamma_d = 10\gamma_Y$ and variable number of stories

Figures 11D and 11H depict the median and 84th percentiles of the peak plastic rotation demands along the height of the steel MRFs with elastic and inelastic panel zones at DBE and MCE, respectively. Superimposed in the same figures are the rotation demands at which the onset of local buckling at the beam ends is anticipated. The results suggest that steel MRFs designed with elastic panel zones are likely to experience local buckling at their beam ends even at modest lateral drift demands associated with DBE seismic events. Conversely, steel MRFs with a balanced beam-to-column connection design are only expected to experience flexural yielding at their beam ends even at an MCE seismic event, thereby minimizing the likelihood of structural repairs for the same intensity. However, one aspect to be evaluated is the likelihood of fractures at the bottom flanges of steel beams due to panel zone kinking.

At DBE seismic intensities, the simulation results confirm that none of the steel MRFs experienced beam fractures regardless of the targeted panel zone inelastic shear distortion demands and the associated number of steel MRF bays. Therefore, our focus hereinafter is at seismic intensities associated with a 2% probability of exceedance earthquakes. Naturally, steel MRFs with a lesser number of bays feature deeper beams to achieve the targeted lateral drift requirements by current seismic standards. It is generally known that beam-to-column connections featuring deep beams (i.e., depths larger than 500mm) are more prone to beam flange fractures. Figure 12 shows the likelihood of having one or more beam end fractures per MRF (noted as “fractures” hereinafter) for variable degrees of inelasticity in the panel zone joints (Figure 12A) and MRF heights (Figure 12B). Figure 12A underscores that for the MCE, no beam fractures are anticipated...
for panel zone designs that respect the current specifications. For highly inelastic panel zone designs, the chance of getting up to four fractures is less than 20% for the four-story steel MRF. On the other hand, Figure 11H suggests that at least half of the steel beams of the MRF with elastic panel zone designs will experience local buckling within their dissipative zones. Similar observations hold true for the 8- and the 12-story MRFs (see Figure 12B). From the same figure, taller steel MRFs experience significantly less beam fractures compared to the low-rise ones.

6.3 Engineering demand parameter hazard curves

To further explore the potential benefits of panel zone shear yielding on the seismic response of steel MRFs, in this section, global and local EDP hazard curves are developed within the methodological framework of Performance-based Earthquake Engineering. The EDP hazard curves are computed according to Equation (6). To compute the annual rate of exceeding a certain EDP value, $EDP_i$, the $EDP_i$ fragility curves are numerically integrated over the hazard curve at $d\lambda(im)$ increments. The $EDP_i$ fragility curves are then developed according to Equation (7),

\[
\lambda_{EDP} (EDP_i) = \int_0^\infty P[EDP > EDP_i|im] \cdot |d\lambda(im)| \tag{6}
\]

\[
P[EDP > EDP_i|im] = \left\{ \begin{array}{ll}
P[EDP > EDP_i|NC,im] \cdot (1 - P[C|im]) + P[C|im] & \text{if } EDP_i \text{ is collapse-related,} \\
\sum [EDP > EDP_i|im] / N_r GMs & \text{otherwise} \end{array} \right. \tag{7A}
\]

\[
P[EDP > EDP_i|im] = \left\{ \begin{array}{ll}
P[EDP > EDP_i|NC,im] \cdot (1 - P[C|im]) + P[C|im] & \text{if } EDP_i \text{ is collapse-related,} \\
\sum [EDP > EDP_i|im] / N_r GMs & \text{otherwise} \end{array} \right. \tag{7B}
\]

where $P[EDP > EDP_i|NC,im]$ is the probability of exceeding $EDP_i$ given no collapse at a targeted $im$, which is calculated by assuming the empirical cumulative distribution; and $N_r GMs$ is the number of considered ground motions. Equation (7A) assumes that the probability of exceeding $EDP_i$ at collapse equals one, meaning $P[EDP > EDP_i|C,im] = 1$. This assumption holds true for EDPs that reach ‘infinite’ values when a steel MRF collapses. However, this is not the case for other EDPs, such as the absolute accelerations that saturate once steel MRFs exhibit inelastic behavior. The same holds true for the peak $\gamma/\gamma_y$, EDP (see Figures 8B and 13); hence in these cases, Equation (7B) is utilized. This is illustrated in Figure 13B, where the number of ground motions that lead to peak $\gamma/\gamma_y$ higher than (peak $\gamma/\gamma_y$) are divided by the total number of ground motions for each $im$.

Figure 14 shows the computed EDP hazard curves of the four-story, three-bay steel MRFs. The EDPs of interest are the residual SDRs (see Figure 14A), the peak $\gamma/\gamma_y$ (see Figure 14B), the number of beam end fractures (see Figure 14C) and the number of beam end local buckles (see Figure 14D) per steel MRF. The mean annual frequency of the DBE, $\lambda_{DBE}$, and that of the MCE, $\lambda_{MCE}$, are superimposed as a reference in these figures based on the 475 and 2475 year return periods, respectively. Referring to Figure 14A, steel MRF designs with panel zones achieving $10\gamma_y$ to $15\gamma_y$ experience at least 50% less residual SDR than their code-compliant counterparts. Figure 14B suggests that, at DBE, steel MRFs reach the anticipated $\gamma_d$ only when elastic panel zones are employed. Conversely, steel MRF designs with inelastic panel zones.
do not generally exceed inelastic distortions of more than $7 - 8 \gamma_y$. The improved seismic performance of MRFs designed with panel zones reaching $10 - 15 \gamma_y$ is highlighted in the above discussion. For panel zone design targets of $10 \gamma_y$, even when the beam-to-column relative expected strength is not as anticipated due to the material variability of the adjoining steel members, the structural performance is not impaired. Therefore, some variability in the steel material properties of the respective members is not considered to be detrimental to the overall beam-to-column connection performance under seismic loading.

At this level of inelastic shear distortions there are no documented fractures in prequalified beam-to-column connections, as highlighted in Figure 14C. At MCE, panel zones reach distortions close to their design targets. In this case, up to two beam bottom flange fractures per MRF may be expected. Noteworthy stating that at DBE, steel MRF designs with $\gamma_d = \gamma_y$ lead to local buckles within the beam ends, whereas this is not the case for steel MRF designs with inelastic panel zones. At MCE, the number of beam end local buckles is nearly half when steel MRF panel zones feature $\gamma_d = 15 \gamma_y$ designs compared to those for steel MRFs with $\gamma_d = 10 \gamma_y$.

Figure 15 summarizes expected global and local EDPs of interest at DBE for the three-bay MRFs, as extracted from the developed EDP hazard curves. The results suggest that residual SDRs decrease in steel MRFs with inelastic panel zones regardless of the steel MRF height (see Figure 15A). For instance, in steel MRF designs with $\gamma_d = 10 - 15 \gamma_y$, there is at least a 50% reduction in residual SDRs compared to their counterparts with $\gamma_d \leq 4 \gamma_y$. According to Figure 15B, code-compliant panel zone designs (i.e., $\gamma_d \leq 4 \gamma_y$) are likely to achieve $\gamma_d$ at DBE. However, steel MRFs with panel zones designed to attain $\gamma_d = 10 - 15 \gamma_y$ reach distortions of up to $5 - 8 \gamma_y$. These findings hold true for all the examined steel MRFs regardless of their height. Moreover, beam flange fractures are unlikely to occur at DBE for all the examined steel MRFs. On the other hand, designs where panel zones are designed to remain elastic exhibit local buckling in 3–10 beam-to-column connections (see Figure 15C), thereby increasing the anticipated repair costs and potential downtime in the aftermath of earthquakes.

Figure 16 compares the expected number of fractures at the bottom beam flange of the steel MRF connections at MCE for the three- and the five-bay steel MRFs. The designs with $\gamma_d \leq 10 \gamma_y$ do not experience any fractures, regardless of the number of bays in the steel MRFs. The three-bay designs with $\gamma_d = 15 \gamma_y$ experience one to four beam flange fractures per
SUMMARY AND CONCLUSIONS

This paper provides quantitative knowledge on important seismic response characteristics of steel frame buildings with MRFs with inelastic panel zone joints. For this reason, two beam-to-column connection modeling approaches are proposed for simulating the hysteretic response of steel MRF beam-to-column web panel zone joints. The first approach involves shell elements, whereas the second one features a macro-model. The former captures the panel zone yielding evolution, the cyclic hardening, and the axial-to-shear load interaction. The latter utilizes an improved tri-linear backbone curve to simulate the panel zone response and assumes an effective panel zone uniform yielding, while it disregards the cyclic hardening phenomena and the shear-to-axial load interaction. Both models consider the shear and bending panel zone deformation modes, which is a limitation of available panel zone models in the literature. Validations with available experimental data suggest that both modeling approaches accurately represent the panel zone hysteretic response in steel MRFs. Moreover, the seismic response of steel MRFs is not practically affected by the employed modeling approach. A methodology is also proposed to consider the fracture potential in steel MRF beam-to-column connections with highly inelastic panel zones (i.e., shear distortions higher than $10\gamma_y$, where $\gamma_y$ is the panel zone distortion at yield).

Nonlinear static and dynamic analyses are conducted to evaluate the seismic demands and the collapse risk of prototype steel buildings with MRFs as their primary lateral load resisting system. The archetype steel buildings feature welded beam-to-column connections with variable targeted inelastic shear distortions. Geometric parameters that are examined involve the number of stories as well as number of bays in steel MRFs. The prototype buildings are designed based on ASCE/SEI 7–16 and AISC,1,17,41 except for the panel zones that comprise code-compliant as well as highly inelastic panel
Nonlinear static analyses demonstrate that the static overstrength factor of steel MRFs is not practically influenced by the level of inelastic deformations within the panel zone joints. Designs with highly inelastic panel zones (i.e., inelastic shear distortions of $10 - 15\gamma_y$) generally increase the period-based ductility of steel MRFs by 20%–30% relative to their elastic panel zone counterparts.

Low-rise steel MRFs with highly dissipative panel zones (i.e., $10 - 15\gamma_y$) demonstrate up to 30% lower collapse risk than that of conventionally designed steel MRFs ($\gamma \leq 4\gamma_y$). The results suggest that the number of beam end local buckles per MRF is minimal in this case, contrary to those in steel MRFs featuring elastic panel zone designs. In steel MRFs with inelastic panel zones, these attain $7 - 8\gamma_y$ even when the design targets $10 - 15\gamma_y$, because the respective beams do not reach their capping moment at a design basis earthquake. None of the examined steel MRFs exhibited any connection fractures at the same seismic intensity.

At seismic intensities associated with a maximum considered earthquake, the expected residual SDRs range from 1%–2% rads for steel MRF designs with highly inelastic panel zones, which are two times lower than corresponding results with code-compliant steel MRFs (i.e., $\gamma \leq 4\gamma_y$). The above findings are important when considering the collapse risk during mainshock-aftershock earthquake series.

The simulation results reveal that beam-to-column connections that provide good balance between the inelastic deformations occurring in the panel zone and the adjoining steel beams, these do not experience local buckling, contrary to the elastic panel zone designs. It is also found that the number of beam flange fractures is zero even for steel MRF designs with targeted inelastic panel zone shear distortions of $10\gamma_y$ even for a low probability of occurrence seismic event (2% probability of exceedance over 50 years).

While at rare seismic events (return periods higher than 2475 years), it is probable that the number of beam-to-column connection fractures increases for highly inelastic panel zone designs, at this level of lateral drift demands (6% rads and above), the steel MRF seismic stability is governed by P-Delta effects.
tigations of the proposed design concept, and will also provide documented improvements of the proposed model for system-level building simulations.

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