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Seismic stability of steel moment resisting frames with inelastic panel zones

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ABSTRACT

This paper summarizes preliminary findings of nonlinear simulations aiming to quantify the seismic demands of code-compliant steel MRFs with variable panel zone targeted shear distortions, γ_d. The probability of beam fractures is explicitly considered in the modeling approach. In steel MRF designs with balanced beam-to-column connections (i.e., targeted $\gamma_d = 15\gamma_y$, where γ_y is the panel zone shear distortion at yield), the onset of local buckling in the steel beams does not occur prior to lateral drift demands of 4 to 5% rads, on average. For drift controlled MRFs, this slightly increases their collapse capacity relative to steel MRFs featuring elastic panel zones. This is because P-Delta effects mostly control the dynamic instability of steel MRFs in this case. The simulation results suggest that steel MRF designs with more balanced beam-to-column connections that conform to the current fabrication weld practice do not experience beam fractures at modest drift demands associated with a design-basis earthquake. Moreover, the likelihood of residual story drift ratios along the steel MRF height reduces by 70% relative to steel MRFs that mostly concentrate their inelastic deformations in steel beams.

Introduction

Capacity-designed steel moment resisting frames (MRFs) are designed so as plastic deformations are mostly concentrated in the beams and the first-story column bases. The panel zone participation in energy dissipation is minimal, while beam softening due to local buckling is inevitable. This influences the collapse capacity of steel MRFs. Structural repairs in the respective steel beam ends are likely due to local buckling at modest lateral drift demands [1].

Recent surveys by Skiadopoulos and Lignos [2] on experiments in welded unreinforced flange-welded web (WUF-W) beam-to-column connections highlight a stable overall connection response when panel zones attain targeted inelastic distortions, γ_d of $10 - 15\gamma_y$, where γ_y is the panel zone shear distortion at yield. It was also found that, in the above cases, the probability of connection fracture is less than 5% prior to 4% story drift ratio (SDR). Moreover, the likelihood of connection fractures becomes negligible for connections featuring shallow steel beams (i.e., $d_b < 500mm$).

Figure 1a depicts the hysteretic response of two nominally identical subassemblies tested by Shin and Engelhardt [3]. The only difference between the two cases is the presence of the doubler plates in the former case ($\gamma_d = 2\gamma_y$) and their absence in the latter ($\gamma_d = 12\gamma_y$). In the former case, cyclic deterioration due to local

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buckling is evident at 3% SDR (see Fig. 1b), leading to more than 30% loss in the lateral load carrying capacity of the subassembly. In the latter case, the hysteretic response is very stable until 5% SDR where fracture occurs with the panel zone being the main source of energy dissipation (see Fig. 1c).

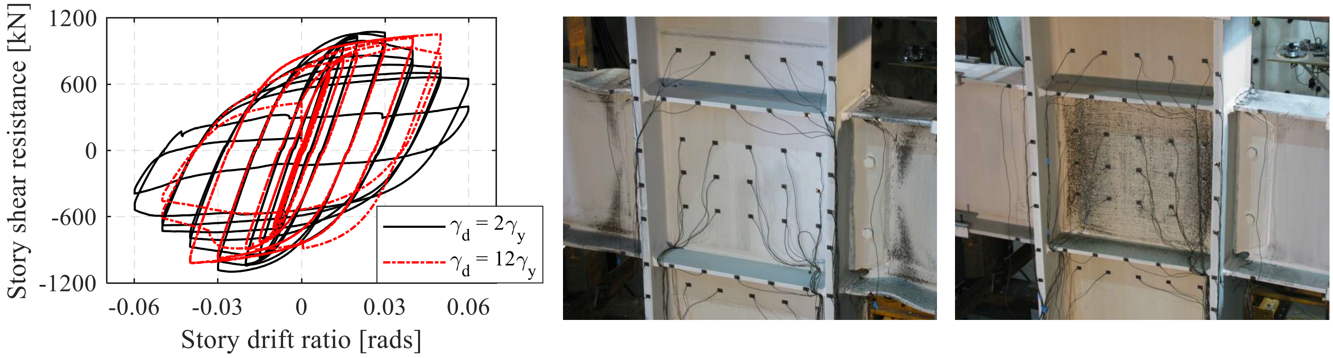

Figure 1. (a) Hysteretic responses of beam-to-column subassemblies featuring $2\gamma_y$ and $12\gamma_y$ panel zone shear distortions; deformed shapes at 3% SDR for the (b) $2\gamma_y$; and the (c) $12\gamma_y$ cases (test data and images were adopted by [3]).

In this paper, the seismic demands and the collapse risk of four-story steel MRFs for varying γ_d and number of bays are identified through nonlinear analysis. Three- and five-bay MRFs are considered to investigate the effect of increased fracture potential associated with connections utilizing deep beams. Panel zones with $\gamma_d = \gamma_y$ as per the AISC specification [4] and $\gamma_d = 12\gamma_y$ that violate the current AISC specifications are considered. Hazard curves for engineering demand parameters (EDP) of interest are developed to assess the seismic demands of the examined steel MRFs within the framework of Performance-Based Earthquake Engineering.

Archetype Steel Building Description and Modeling Approach

The design location of the considered prototype steel building is in urban California (coordinates: 33.996°N, 118.162°W) with soil class D and risk category II. The steel building is shown in Fig. 2a in plan view. It consists of two perimeter 5-bay MRFs (see Fig. 2b), two orthogonal concentrically braced frames (CBFs), and a gravity framing system. A 3-bay steel MRF is also designed. The design is based on the current AISC specifications [4–6] and ASCE 7-16 [7]. Pre-qualified WUF-W beam-to-column connections are considered. For all MRFs, the strong-column/weak-beam ratio ranges from 1.4 to 2.2.

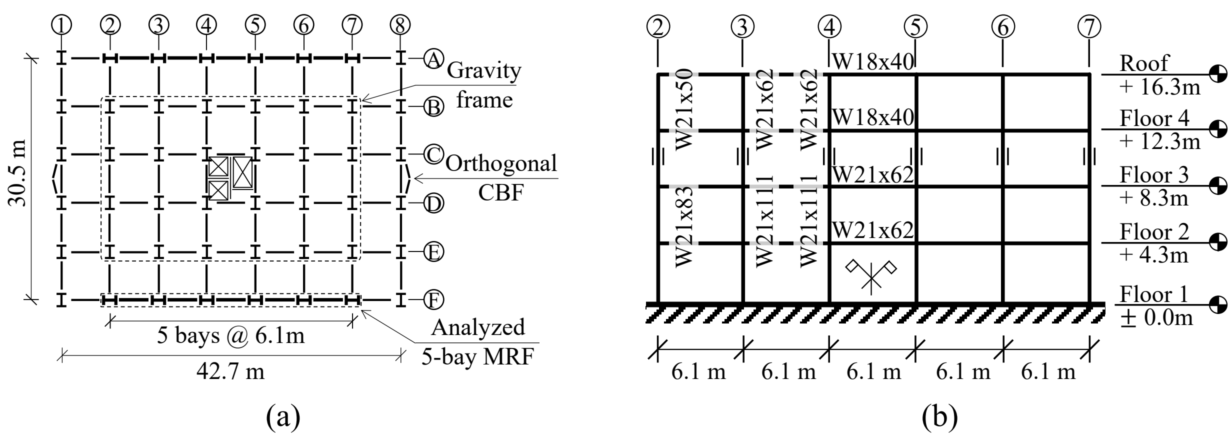

Figure 2. Five-bay, four-story steel MRF: (a) typical plan view; and (b) elevation view.

The analyzed two-dimensional (2D) MRFs are modeled in the Open System for Earthquake Engineering Simulation (OpenSees) [8]. Concentrated plasticity models are employed, with rotational zero-length deterioration elements to simulate structural damage. The parameters of these models are defined according to predictive equations for steel columns [9] and beams [10]. For the panel zones, the parallelogram model by

Gupta and Krawinkler [11] is adopted. However, the tri-linear backbone curve by Skiadopoulos et al. [12] is utilized to idealize the inelastic behavior of the panel zones. A leaning column that is connected to the MRF is considered to account for the destabilizing effects of the gravity load. The probability of connection fracture is explicitly considered in the modeling strategy. For this purpose, the model updating technique of OpenSees is utilized in which the beam rotational zero-length element loses its moment-carrying capacity once the probability of connection fracture, P, exceeds a threshold value and the bottom beam flange is in tension. The probability of connection fracture is computed according to [2].

Pushover Analysis Results

Nonlinear static analysis (i.e., pushover) based on a first-mode lateral load pattern is performed to quantify the effect of γ_d and number of MRF bays on the static overstrength factor, Ω, and on the period-based ductility factor, μ_T, as per the FEMA P-695 methodology [13]. These parameters together with the first-mode periods of the MRFs are summarized in Table 1. The first-mode period of the weaker panel zone design is marginally higher compared to that of the stronger one. The stiffness of the former is almost 5% smaller compared to the latter. The number of bays does not practically affect the first-mode period of the steel MRFs. The variation of Ω with respect to γ_d is insignificant. As the number of bays increase, Ω tends to increase as well, due to the higher redundancy of the 5-bay steel MRFs relative to the 3-bay ones. For the weaker panel zone design cases, μ_T increases by 30% due to the delay of the onset of beam local buckling.

Table 1. First-mode periods and global performance factors.

	$\gamma_d = \gamma_y$		$\gamma_d = 15\gamma_y$	
	3-bay	5-bay	3-bay	5-bay
T_1 (s)	1.28	1.28	1.32	1.32
Ω	2.3	2.6	2.2	2.5
μ_T	4.0	3.7	5.3	4.9

Figure 3a shows the pushover curves of the analyzed cases. The base shear of the steel MRFs is normalized with respect to half of the seismic weight, W. The peak base shear does not deviate by more than 10%, regardless of the panel zone weakness. Figure 3b depicts the maximum beam and first-story column plastic rotation distribution at each floor, $\theta_{pl.,max}$ at 3% roof drift ratio (see Fig. 3a). The pre-capping plastic rotation, θ_p, is superimposed for each floor according to [9,10]. In steel MRFs featuring strong panel zone designs, the two first floor beams experience local buckling. Conversely, in steel MRFs with a balanced beam-to-column web panel design, none of the steel beams experience nonlinear geometric instabilities (see Fig. 3b).

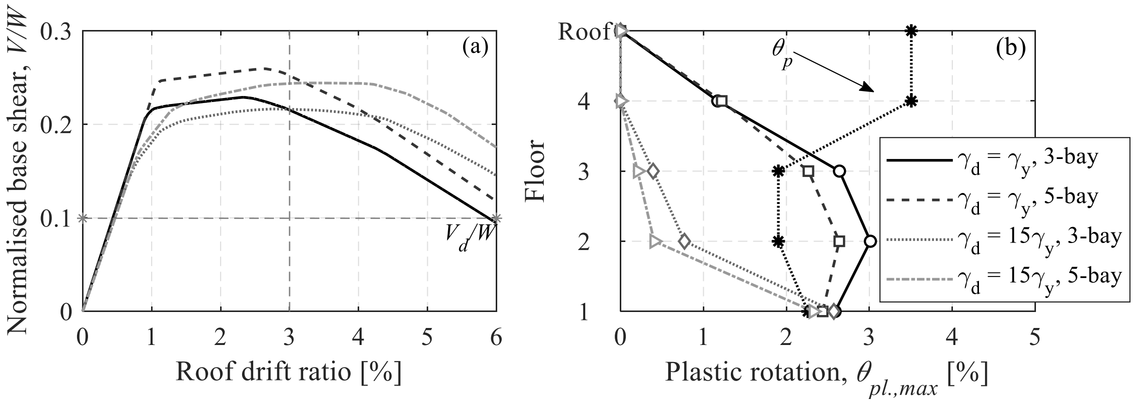

Figure 3. Pushover analysis results: (a) Base shear versus roof drift ratio; and (b) peak plastic rotation along the building height.

Nonlinear Response History Analysis

Incremental dynamic analysis is conducted to evaluate the effect of panel zone weakness on the collapse risk of the MRFs. For this reason, the set of 44 far-field ground motions of FEMA P695 [13] is used. The collapse fragility curves of the MRFs are depicted in Fig. 4a. The first-mode, 5% damped spectral acceleration, $S_a(T_1, 5\%)$ is utilized as an intensity measure in this case. The median collapse intensity of the stronger panel zone designs is about 10% higher than that of the steel MRFs with the weaker panel zones. P-Delta effects strongly influence the collapse capacity of the steel MRFs, because all designs are drift-controlled. Figure 4b shows the median peak SDRs along the steel MRF heights at the last stable point of the IDA curves for the examined cases. The peak SDR distribution is fairly uniform regardless of γ_d. This suggests that steel MRF designs with weak panel zones are not necessarily prone to the formation of soft-story collapse mechanisms.

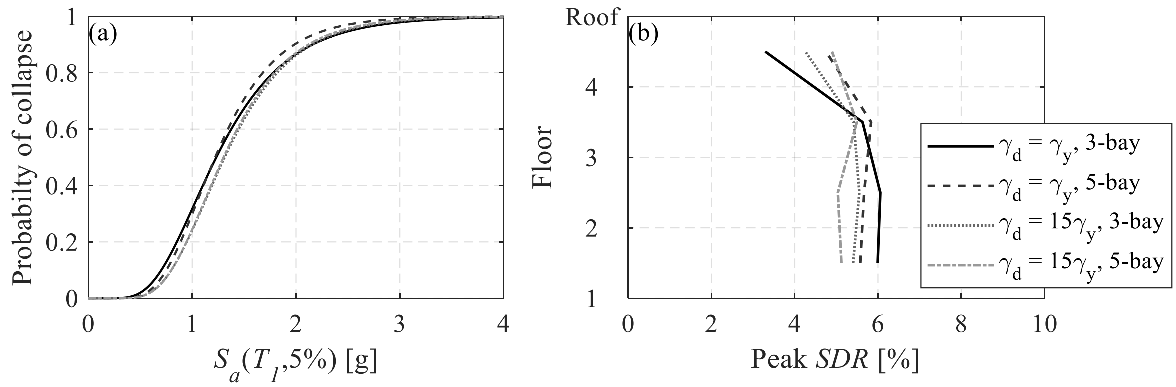

Figure 4. (a) Collapse fragility curves; and (b) median peak story drift ratio profile near collapse.

From a structural repairability standpoint, the annual frequencies of residual story drift ratio ($RSDR$) and beam bottom flange fracture occurrence are quantified according to the approach by [14]. The EDP hazard curves are depicted in Fig. 5. For the design basis earthquake (DBE), steel MRFs with strong panel zones are likely to experience 70% higher $RSDRs$ than their weak panel zone counterparts. Moreover, the simulation results demonstrate that beam fractures are not likely, even in cases that panel zones attain $15\gamma_y$. For a maximum considered earthquake (MCE), there are two beam fractures per MRF for the 3-bay MRF whereas there are no fractures for the 5-bay MRF. This is because the latter features shallower beam depths than the former.

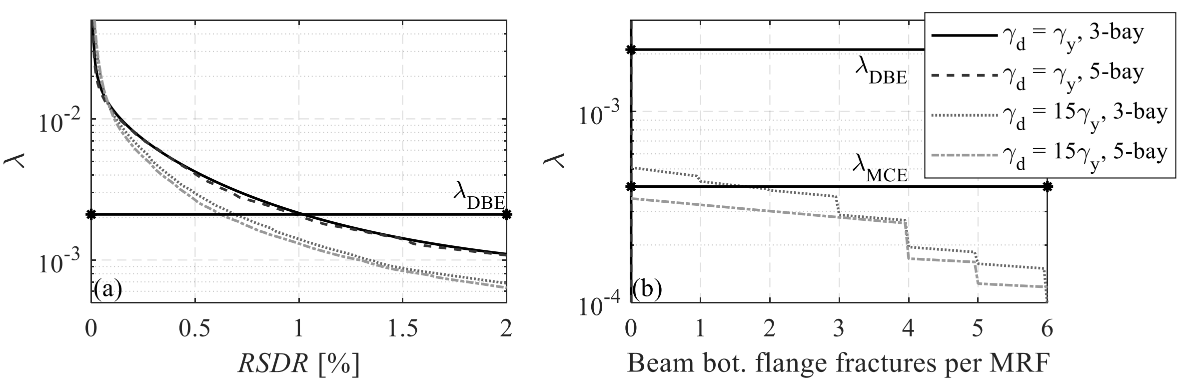

Figure 5. EDP hazard curves: (a) $RSDR$; and (b) beam bottom beam fractures per MRF.

Conclusions

This study quantified the seismic demands of four-story MRFs with targeted panel zone shear distortions, γ_d. The steel MRF designs involve 3- and 5-bays. The results suggest that MRFs with highly inelastic panel zone joints (i.e., $\gamma_d = 15\gamma_y$), are not necessarily prone to soft-story mechanisms. Moreover, the current design practice leads to 70% increased residual story drift ratio at a design basis earthquake. This is attributed to the onset of beam local buckling at modest lateral drift demands (i.e., 1.5-2% rads). The simulation results suggest that modern steel MRFs with welds that conform to the current standards do not experience fractures during a maximum considered earthquake when they feature beam sizes with depths less than 500mm.

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