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Structural modeling and ground motion selection for risk assessment of pre-Northridge welded steel moment frames

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ABSTRACT

Nonlinear response history analyses are the most reliable tool for quantifying the earthquake performance of structures and inform risk management decisions like acquiring insurance or planning a structural retrofit. Constructing these models requires assumptions about the structure and its earthquake hazard that are normally made based on a limited number of analyses or expert opinions. This paper summarizes a sensitivity study that systematically evaluates the effects of the most common assumptions for modeling pre-Northridge welded steel moment frames (WSMF) on robust risk metrics. These assumptions include choosing a connection modeling approach and defining the target parameters for ground motion selection and scaling. The results of this sensitivity study support practical recommendations for applying the full performance-based earthquake engineering framework as implemented in FEMA P58 to tall WSMF buildings.

Introduction

The skylines of major cities on the west coast of the United States are dominated by buildings that rely on older welded steel moment frames (WSMFs) for lateral load resistance [1–3]. The 6.7M_w Northridge (1994, California, US) and 6.9 M_w Kobe (1995, Japan) earthquakes demonstrated that WSMF buildings are prone to experiencing premature brittle fractures of beam-to-column joint welds [4] under moderate to severe earthquake ground motions. Improving their inadequate performance can require complex retrofit interventions that are expensive and disruptive to building function; thus, engineers need to perform rigorous performance-based earthquake engineering assessments to quantify the building's risks and justify any retrofit action [5,6].

The main tools to quantify risk and design retrofit interventions are high-fidelity finite element models that simulate buildings' response to earthquakes using nonlinear response history analyses. Constructing these models requires a significant number of assumptions to model the structure and its earthquake hazard with a set of ground motion accelerograms. These assumptions are often decided based on expert opinions due to lack of guidelines in the literature. For instance, the FEMA 355F [7] state-of-the-art report lists recommendations for modeling beam-to-column connections, yet it does not include guidance for choosing the connection parameters and accounting for their uncertainty. Consideration of uncertainties is also absent in the more recent ASCE/SEI 41 [5] specification for seismic evaluation and retrofit of existing buildings. Similarly, the ground motions at a site could be selected and scaled using

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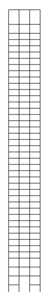
different targets like the Uniform Hazard Spectrum (UHS), a conditional mean spectrum [8], a conditional spectrum [9], or another prescribed spectrum. Furthermore, one could decide to impose further constraints in ground motion selection, including strong motion duration [10] and velocity pulses [11]. The final choices of structural modeling and ground motions could significantly alter the risk and retrofit design.

To inform these decisions, this paper summarizes a sensitivity study on the effect of the most common assumptions for modeling and selecting ground motions in risk assessments of pre-Northridge tall WSMFs. The results are distilled down into concrete recommendations for reliable and efficient quantification of the collapse safety and loss risk, i.e., probability of collapse in 50 years ($P_c^{50\text{years}}$) and the expected annual loss ratio (EAL), using the FEMA P58[12] methodology implemented in the NHERI SimCenter's Pelicun package[13].

Building archetypes and structural modeling

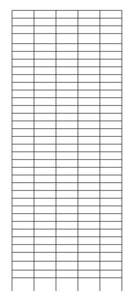
The sensitivity study is performed using two frame archetypes, shown in Fig. 1a and Fig. 1b, which are modeled after existing buildings in San Francisco and are generally representative of frame designs of the late-1960's through mid-1970's. Both archetypes comply with the applicable building code in effect at the time of their design, representing space frame configurations of similar age and height, but with different dynamic properties. Based on the calculated building periods, building A (Fig. 1a) is a significantly more flexible system than building B (Fig. 1b), where building B has both proportionally larger section sizes and more framing bays than building A.

Year	1971
No. stories	42
Height	474 ft
No. bays	3
Frame	Space
T_1	5.10 s

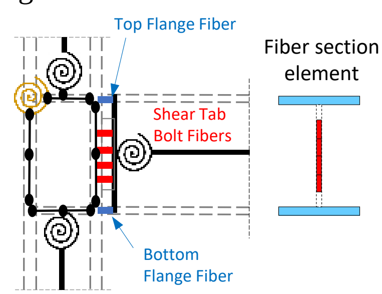


(a)

Year	1971
No. stories	35
Height	472 ft
No. bays	5
Frame	Space
T_1	3.30 s



(b)



(c)

Figure 1. Tall building archetypes of similar completion year and height: (a) building A (flexible archetype) and (b) building B (stiff archetype). (c) Structural model of welded connections using a fiber section with *SteelFractureDI* for the flanges.

Each frame was modeled in OpenSees [14] using elastic elements with nonlinear springs to represent plastic hinging at the ends of the beams and columns, along with Krawinkler's [15] model for the panel zones (Fig. 1c). The beam-to-column connection is traditionally represented in the beam's plastic hinge using a prescriptive backbone curve per ASCE/SEI 41 that captures the effect of fracture with a rotation limit that omits the impact of repeated cycles or crack closing. Alternatively, we developed a fiber-section element that simulates the welded flanges with single fibers (shown in blue in Fig. 1c) and the shear tab bolted connection with multiple fibers (shown in red in Fig. 1c). The flange fibers use a phenomenological material model called *SteelFractureDI* that employs a novel fracture mechanics-based criterion, which accounts for ultra-low cycle fatigue effects. This material model explicitly considers both the fracture toughness—measured with a Charpy-V-notch (CVN) test—of the weld metal and the presence of weld defects or initial flaws (a_0). The material model is also equipped with a post-fracture constitutive law that simulates the gapping behavior of fractured flanges that open the crack in tension and resist compression once the crack closes. More details on the building and connection models can be found in [16].

The most important assumptions in the WSMF model relate to selecting the connection model, selecting the weld material properties (e.g. CVN and a_0), and considering the uncertainties in the modeling parameters. In this study, the baseline connection model has fiber-section elements using CVN=12ft-lb and a_0=0.1t_f (t_f, beam flange thickness). We created seven variations of this baseline to quantify the impacts of the aforementioned assumptions. The specific variations are using fiber-sections with (1-2) CVN=20ft-lb or 40ft-lb; (3) 12ft-lb, including modeling uncertainty in fracture prediction (FI UQ); (4) CVN uncertainty as a uniform value per record (CVN uniform + FI UQ), and (5) CVN uncertainty as a unique value per connection per record (CVN by connection + FI UQ). In addition, we included two models where the fiber-section connection element is replaced with either (6) the ASCE/SEI pre-Northridge hinge model or (7) the ASCE/SEI (post-Northridge) non-RBS hinge model. These last two variations represent state-of-practice modeling approaches.

Ground motion sets

Estimating the risk of a building also requires representing the earthquake hazard at the site (San Francisco) with carefully selected and scaled sets of ground motions. In this study, we evaluated the effect of three assumptions in ground motion selection: the target spectra, the conditioning period, and the strong-motion duration.

To this end, we selected sets of 70 ground motions at multiple intensities per building based on four criteria: (1) conditional spectra at the fundamental period of the building (CS T_c=T_1), (2) conditional spectra at 1.0s to capture higher-mode effects (CS T_c=1.0s), (3) uniform hazard spectra (UHS), and (4) spectrally-equivalent long duration sets conditioned on the fundamental period of the building (CS T_c=T_1 & $D_{s5\text{-}75}$). The spectrally equivalent long-duration sets have a duration distribution that is similar to that expected at sites with a hazard dominated by subduction interface earthquakes like Eugene, OR, or Seattle, WA, (significant duration from 30 to 45s), while the other sets have durations similar to those expected in San Francisco (significant duration from 10s to 20s) [17].

Results

Shown in Fig. 2 are the collapse fragilities for the building archetypes computed with multi-stripe analysis using each of the four ground motion sets. The figure also shows the two most common collapse mechanisms for each frame. These results show the drastically different behavior of these two archetypes where building A is significantly more vulnerable than building B. For buildings A and B respectively, the baseline model using the reference ground motions (blue curve) have collapse risks of $P_c^{50\text{years}}$ equal to 0.14 and 0.04 and loss risks of EAL equal to 0.48% and 0.31% (as a fraction of replacement value). The calculated collapse risks are significantly larger than the $P_c^{50\text{years}}$ value of 0.01 assumed in developing the risk-targeted MCE$_r$ maps of ASCE 7[18], and the losses are similar to values of 0.6% to 1.0% that have been estimated for code-conforming modern buildings [19,20].

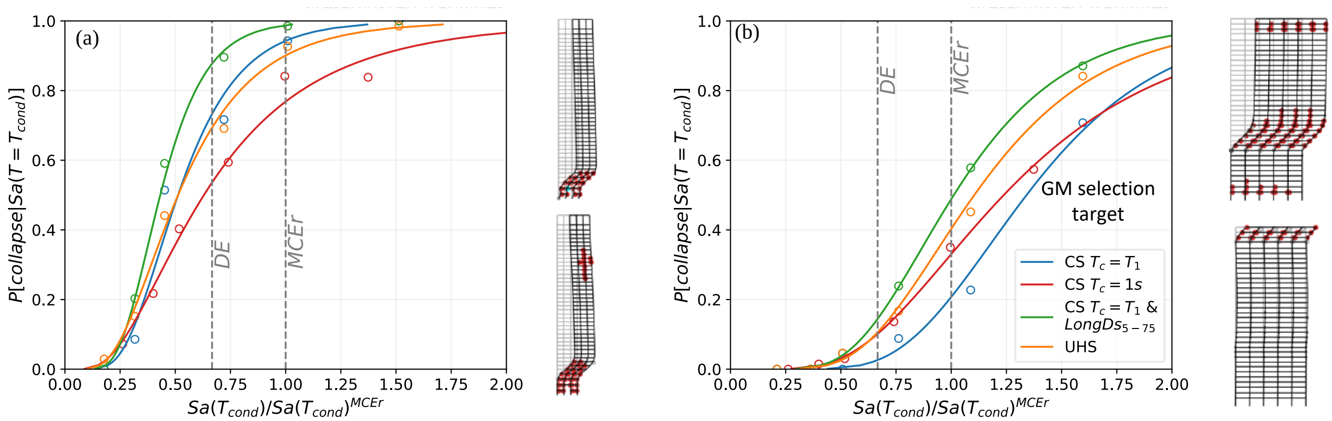


Figure 2. Collapse fragility functions varying the ground motions selection parameters and most common collapse mechanisms for frames (a) building A and (b) building B.

This drastic difference in collapse safety of the buildings is evident in the different collapse modes, shown in Fig. 2. The collapse behavior of building A is dominated by P-Delta effects, due to the low stiffness of the frame, whereas the collapse modes of building B are more sensitive to the distribution of connection fractures, whose locations are more varied. Since the response of building A is dominated by consistent P-Delta effects, the resultant collapse fragilities calculated using the four ground motion sets are very similar in the lower tail of the distribution, shown in Fig. 2a where the ground motion intensities are normalized by the MCE_r intensity. In contrast, the fragilities vary at higher intensities, where the CS T_c=1.0s ground motion set yields the lowest collapse risk, while the CS T_c=T_1 & $D_{s5\text{-}75}$ yields the highest. By contrast, building B experiences a shift in the median collapse capacity with each ground motion selection criterion, indicating that the selection targets are more important for buildings that experience more varied collapse mechanisms under different earthquake input motions.

Summarized in Fig. 3 are the differences in the $P_c^{50years}$ and the EAL results between the baseline model and other model and ground motion variations. This figure shows that the selection of the connection model and its parameters (CVN and a_0) significantly impacted the resultant risk metrics, whereas the modeling uncertainties (i.e., CVN and fracture prediction variabilities) have much smaller effects on the risk metrics. Fig. 3 also shows that ground motion duration is as important as spectral shape for building B whose behavior is more sensitive to connection fracture (as opposed to overall P-Delta instabilities). Furthermore, these results support previous findings that the conditioning period has a very small effect on annualized collapse metrics like $P_c^{50years}$ (less than 10% variations)[21] but demonstrates that economic loss still considerably changes with the conditioning period even for annualized metrics like EAL (40% variations). Thus, the conditioning period must be carefully selected for loss assessment of WSMFs.

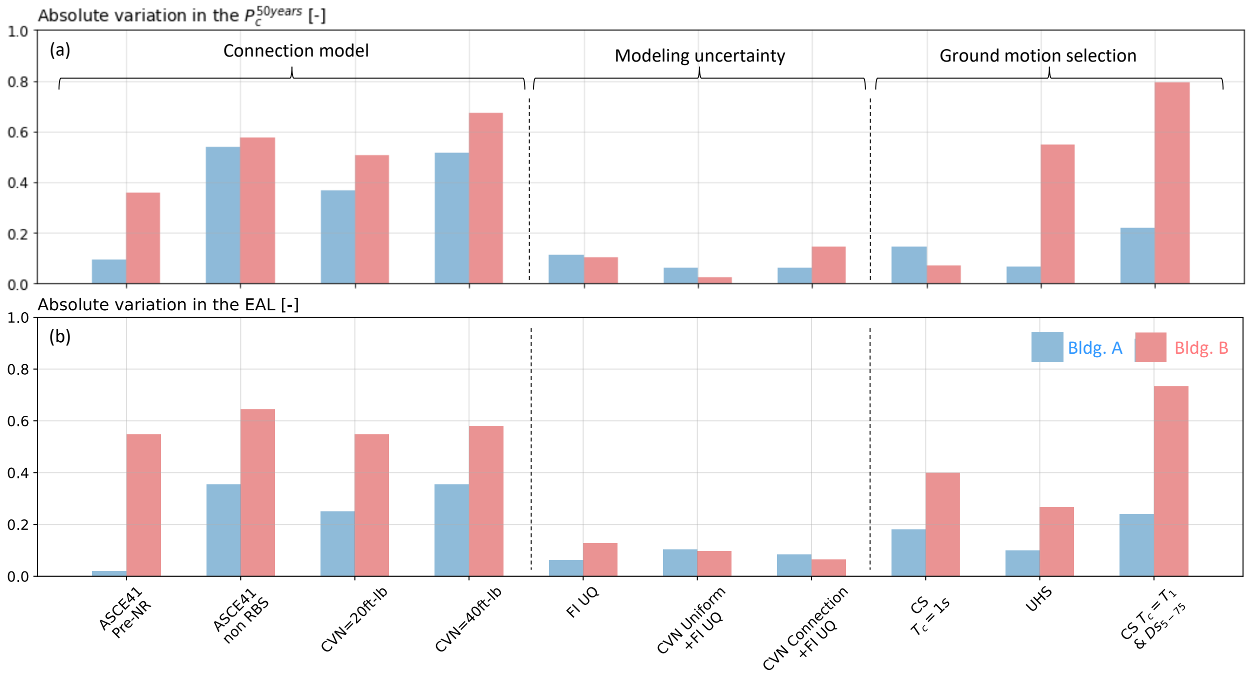


Figure 3. (a) Variation in the risk of collapse and (b) Variation in the expected annual loss with respect to the baseline model and reference ground motion sets caused by the 10 parameters considered in the sensitivity study for buildings A and B

Conclusions

The sensitivity study presented in this paper provides practical guidelines for constructing nonlinear models of welded steel moment frames suitable for risk quantification. The study focused on three assumptions that are subjectively selected in conventional practice: (1) choice of the connection model and median material properties; (2) handling of modeling uncertainty in the connection modeling; and (3) ground motion selection criteria. The results show that the choices of the connection model and median material properties significantly change the risk and economic loss of tall WSMFs. In contrast, explicit quantification of the uncertainty in fracture prediction has a negligible effect on the building's risk.

Ground motion selection criteria are as important as selecting the proper connection model for well-proportioned frames that are sensitive to the number and locations of fractured connections. Moreover, these results provide evidence that ground motion duration is as important as spectral shape for simulating WSMFs using high-fidelity models that capture low-cycle fatigue demands. However, the ground motion selection is less relevant for very flexible frames with dynamic instability driven by first-mode P-Delta effects. As with previous research, we found that the choice of conditioning period has a negligible impact on the collapse risk. Unlike other studies, we show, for the first time, that this conclusion is not true for the economic loss, even for annualized metrics.

Acknowledgments

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