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AN EFFICIENT SEISMIC INTENSITY MEASURE FOR SEISMIC RISK ANALYSIS OF STRUCTURES

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ABSTRACT

In a seismic fragility or seismic margins assessment, the estimated structural system response is conditioned on a measure of seismic intensity. The U.S. Geological Survey currently stipulates the site-dependent seismic hazard by mapping the spectral acceleration at a frequency of 2% probability of being exceeded in 50 years. The scatter in structural responses at a given intensity measure (IM) due to record-to-record variability in ground motion ensembles used in nonlinear time history analysis of structures provides a relative measure of the efficiency of an IM. A number of investigators have noted recently that the spectral acceleration, while an improvement over the ground motion-based IMs, is still relatively inefficient when the structure responds nonlinearly and softens during earthquake ground motion. This issue is addressed in this paper with the use of a previously proposed IM that accounts for period softening of the structure. The parameters of this IM are optimized to minimize the uncertainty in probabilistic seismic demand models for typical three-, six-, and nine-story reinforced concrete frames in the Central and Eastern U.S. Ensembles of synthetic earthquake ground motions that were developed specifically for the New Madrid Seismic Zone are used in finite-element based nonlinear dynamic time history analyses. Seismic fragilities that are subsequently derived for the three concrete frames are compared with those previously developed using the spectral acceleration as an IM, and their implications for seismic risk assessment are discussed.

Introduction

Seismic risk assessment requires probabilistic estimation of the safety and performance of buildings and other civil infrastructure under uncertain future seismic events. Several disciplines, including engineering seismology and geology, soil dynamics, structural mechanics and dynamics, building economics are involved in this process (Chandler and Lam 2001). To incorporate all the information from these distinct disciplines in a probabilistic assessment

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procedure efficiently, a framework that allows the work of each discipline to be performed separately and subsequently combined for engineering decision is necessary (Ellingwood *et al.* 2007). The seismic fragility, which is the structural engineering component of this framework, is described by the conditional probability that the structural capacity, C, fails to resist the structural demand, D, given the seismic intensity (hazard), SI, and is commonly modeled by a lognormal cumulative distribution function:

$$P\left[C < D \middle| SI = x\right] = 1 - \Phi\left[\frac{\ln\left(\hat{C}/\hat{D}\right)}{\sqrt{{\beta_{D|SI}}^2 + {\beta_C}^2 + {\beta_M}^2}}\right] \tag{1}$$

where $\Phi[\ ]$ is the standard normal probability integral, $\hat{C}$ is the median structural capacity, associated with the limit state LS, $\hat{D}$ is the median structural demand, $\beta_{D|SI}$ and β_C denote, respectively, the aleatoric (or inherently random) components of uncertainty in D and C, and β_M is the epistemic (modeling) uncertainty.

In a seismic fragility assessment, the seismic intensity measure (IM) depicts the seismic hazard and the estimated structural system response is conditioned on the IM. The desired IM should be *sufficient* and *efficient*, and have a hazard curve that is relatively easy to compute (Giovenale *et al.* 2004). Given the value of the IM, sufficiency of an IM implies that the structural response is independent of any other ground motion characteristics whereas efficiency of an IM is a measure of the variability of the structural response (Luco and Cornell 2001). The linear-elastic pseudo-spectral acceleration at the fundamental period of a structure with 5% damping is dependent on structural properties as well as ground motion characteristics, and in recent years has been the most commonly used seismic intensity measure. The U.S. Geological Survey (USGS) currently stipulates the site-dependent seismic hazard by mapping the spectral acceleration at a frequency of 2% probability of being exceeded in 50 years.

The scatter in structural responses at a given IM due to record-to-record variability in ground motion ensembles used in nonlinear time history analysis (NTHA) of structures provides a relative measure of the efficiency of an IM. A number of investigators have noted recently that the spectral acceleration, while an improvement over the ground motion-based IMs, is still relatively inefficient when structures respond nonlinearly to earthquake ground motions (Cordova *et al.* 2000; Krawinkler *et al.* 2003; Giovenale *et al.* 2004; Baker and Cornell 2005; Luco and Cornell 2007). This issue is addressed in this paper with the use of an IM that accounts for period softening in structures that respond in the nonlinear range (Cordova *et al.* 2000). The parameters of this IM are optimized to minimize the uncertainty in probabilistic seismic demand models of three-, six-, and nine-story gravity load designed (GLD) reinforced concrete (RC) frames in the Central and Eastern U.S. (CEUS). These demand models, in turn, are used to derive seismic fragilities for seismic vulnerability assessment in the region. Ensembles of synthetic earthquake ground motions that were developed specifically for the New Madrid Seismic Zone (NMSZ) are used in finite-element based nonlinear dynamic time history analyses. Seismic fragilities that are derived for the three concrete frames are compared with those previously developed using the spectral acceleration as the IM (Celik and Ellingwood 2009), and their implications for seismic risk assessment are discussed.

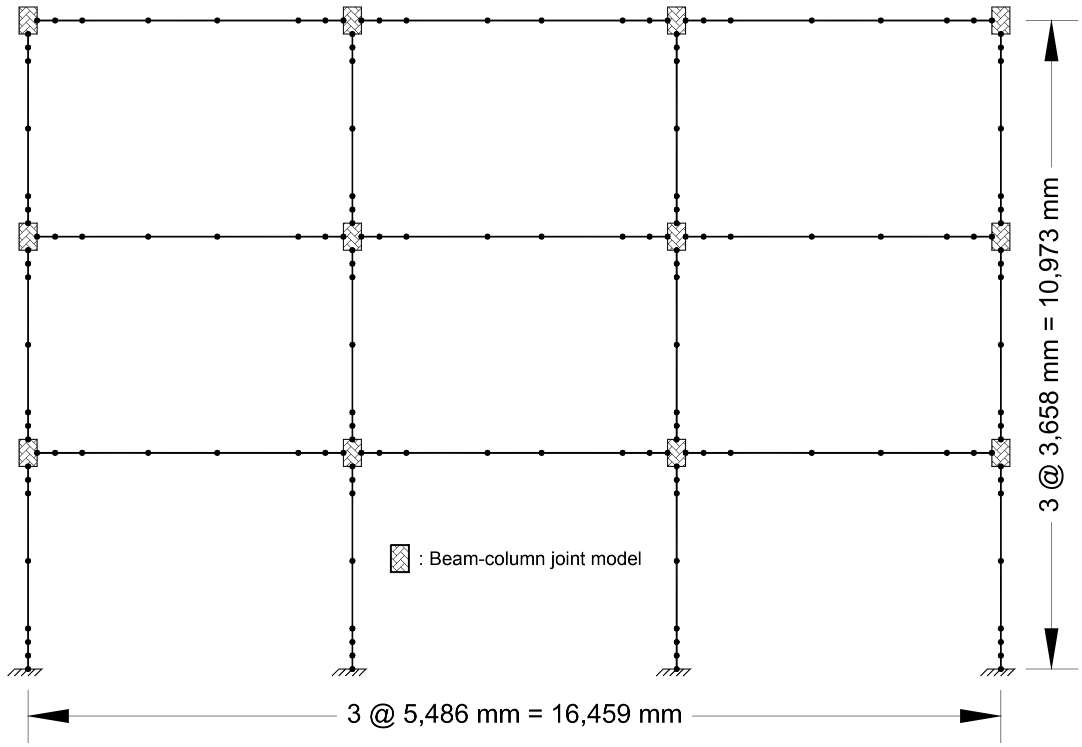


Figure 1. OpenSees model of the three-story frame (1 in. = 25.4 mm).

Finite-Element Structural Models

Finite element analyses of the three-, six-, and nine-story RC frames that typify pre-1990 RC construction practices for low-, mid-, and high-rise RC frames in the CEUS (Celik 2007; Celik and Ellingwood 2009) were performed using OpenSees (McKenna and Fenves 2006). Figure 1 illustrates the structural model of the three-story frame as a typical example; the six- and nine-story frames have the same number of bays, story height, and bay width. OpenSees accounts for geometric and material nonlinearities and, as an open source platform, facilitated the implementation of the beam-column joint model for GLD RC frames (Celik and Ellingwood 2008). The fundamental periods of the frames were 1.1 s, 1.9 s, and 2.8 s, respectively (consistent with the previous research summarized in Celik (2007)). These GLD RC frames were found to be vulnerable to damage from joint shear failures (and excessive joint deformations), beam bottom bar anchorage failures, significant P-Δ effects (due to high flexibility of the frames), and weak column-strong beam effects leading to collapses in a story sway mode under earthquake excitation (Celik and Ellingwood 2008, 2009).

Synthetic Earthquake Ground Motions for the Central and Eastern U.S.

Most seismic fragility analyses of frames in high seismic regions performed previously (*e.g.*, Cornell *et al.* 2002) have utilized natural strong ground motion records in the NTHA. Such records are unavailable for sites in the CEUS due to the infrequent nature of the earthquakes in that region. Accordingly, ensembles of synthetic earthquake ground motions that were developed specifically for the NMSZ under the auspices of the MAE Center by Rix and Fernandez (2006) were utilized in the finite element-based NTHA in this study. The Rix-Fernandez ground

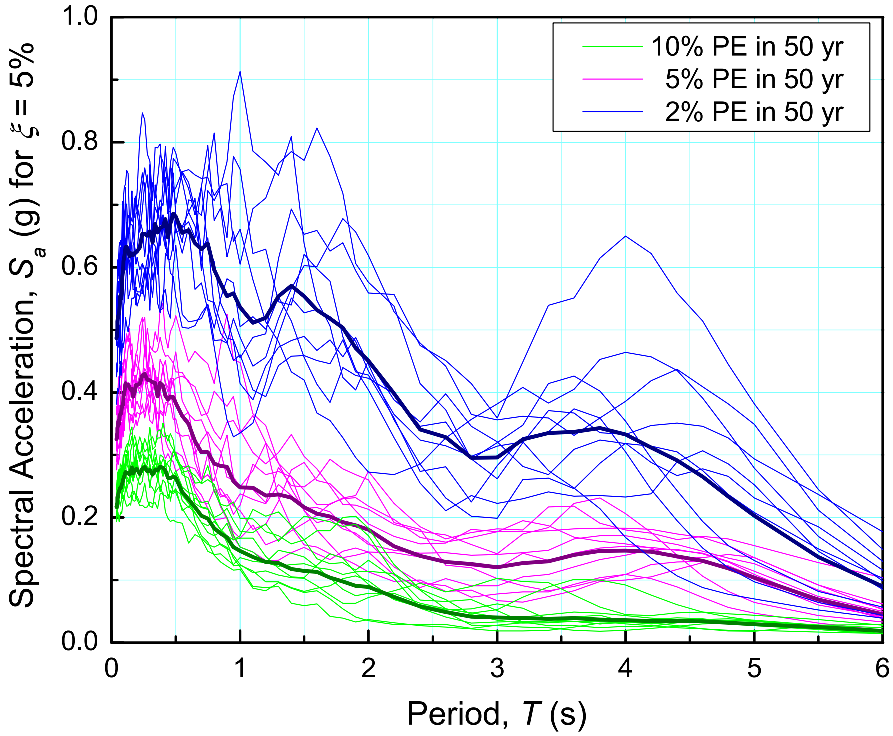


Figure 2. Individual and median response spectra of the Rix-Fernandez ground motions for Memphis, TN (Uplands profile).

motions were developed for three seismic hazard levels: 10, 5, and 2% probability of exceedance (PE) in 50 yr for soil sites in the Upper Mississippi Embayment. For each hazard level, ensembles of 10 ground motions were generated, which subsequently are used in the NTHAs of the GLD RC frames. Figure 2 shows the elastic 5% damped response spectra (spectral accelerations) for the individual synthetic records, and the median response spectrum for each ensemble generated for Memphis, TN.

Optimization of the Intensity Measure — Probabilistic Seismic Demand Models

The seismic demands on the frames were determined using the ensembles of synthetic earthquake ground motions summarized in Fig. 2. The results can be represented by a simple probabilistic demand model relating *D* to the *SI* (Cornell *et al.* 2002):

$$D = a \cdot SI^{b} \cdot \varepsilon \tag{2}$$

where *a* and *b* are constants determined from the analysis of nonlinear seismic demand and ε is a lognormal random variable with median of 1.0 and logarithmic standard deviation $\sigma_{\ln\varepsilon} = \beta_{D|SI}$ depicting the uncertainty in the dependence of *D* on *SI* (or in other words, the contribution to uncertainty in seismic demand due to record-to-record variability (Ellingwood *et al.* 2007, Celik and Ellingwood 2009, 2010)).

In a recently published study (Celik and Ellingwood 2009), which evaluated the seismic risk of GLD RC frames subjected to Mid-America ground motions, the structural demand measure, D, was selected to be the maximum interstory drift angle in the frames, θ_{max}, that occurs during their dynamic response to earthquake excitation, whereas the spectral acceleration at the fundamental period of the frames, $S_a(T_1)$, for 5% damping was adopted as the seismic intensity measure. At lower levels of excitation, the interstory drift provides insight regarding the potential for damage to nonstructural components, while at higher levels it is closely related to structural or local collapse due to excessive P-Δ effects. The choice of $S_a(T_1)$ as an IM is consistent with previous studies and with the specification of seismic hazard by the USGS. On the other hand, shortcomings of using $S_a(T_1)$ as an IM are pointed out by researchers, among them:

(1) The period softening associated with inelastic behavior is ignored (Cordova *et al.* 2000; Krawinkler *et al.* 2003; Giovenale *et al.* 2004; Baker and Cornell 2005; Luco and Cornell 2007);

(2) The contribution of higher modes to the structural response is not considered (Krawinkler *et al.* 2003; Giovenale *et al.* 2004; Luco and Cornell 2007); and

(3) It is not particularly efficient nor sufficient for:

(a) long-period buildings (Shome *et al.* 1998; Luco and Cornell 2007)

(b) soft soil ground motions (Kurama and Farrow 2003; Luco and Cornell 2007) and

(c) near-field ground motions (Krawinkler *et al.* 2003; Kurama and Farrow 2003; Luco and Cornell 2007).

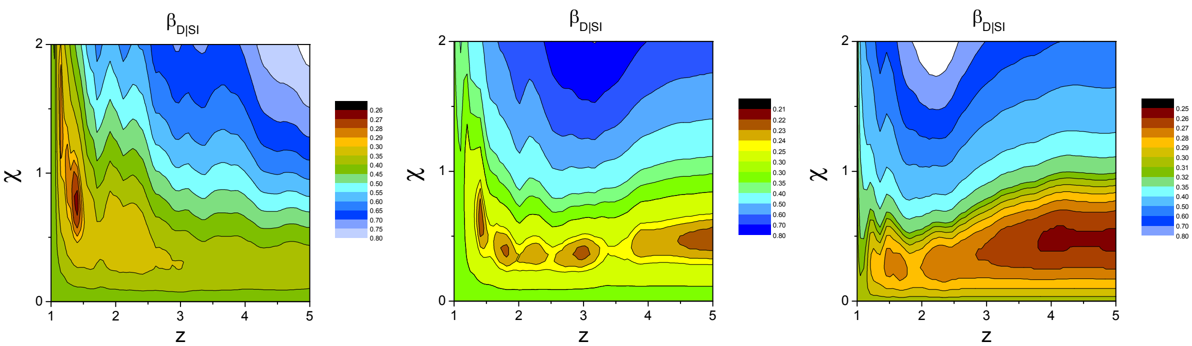
The impact of the choice of the ground motion ensemble and the use of $S_a(T_1)$ as an IM for seismic fragility assessment of GLD RC frames in the CEUS was discussed in Celik and Ellingwood (2009). The inadequacy of the $S_a(T_1)$ in capturing the higher spectral intensities of the Rix-Fernandez ground motions at the lengthened periods of GLD RC frames (*cf.* Fig. 2) was noted. Accordingly, an IM that accounts for period softening in structures (Cordova *et al.* 2000) is used in this study for developing probabilistic seismic demand models (*cf.* Eq. 2) for the three-, six-, and nine-story GLD RC frames in Celik and Ellingwood (2009). This IM is defined by:

$$S_{eff}(T_1) = S_a(T_1) \cdot \left[\frac{S_a(zT_1)}{S_a(T_1)} \right]^{\chi} \tag{3}$$

where z and χ are the parameters that minimize the scatter in structural responses due to record-to-record variability in ground motion ensembles used in NTHA.

Figure 3 displays the optimization of the parameters of $S_{eff}(T_1)$ for each of the three frames. This optimization process yields the following (z, χ) pairs: (1.4, 0.75), (1.4, 0.6), and (1.65, 0.25) respectively for the three-, six-, and nine-story frames. Setting z = 1.5 and χ = 0.5, the IM is defined as the geometric mean of the $S_a(T_1)$ and $S_a(1.5T_1)$:

$$S_{eff}(T_1) = \sqrt{S_a(T_1) \cdot S_a(1.5T_1)} \tag{4}$$



(a) 3-story (b) 6-story (c) 9-story

Figure 3. Optimization of the parameters of $S_{eff}(T_1)$.

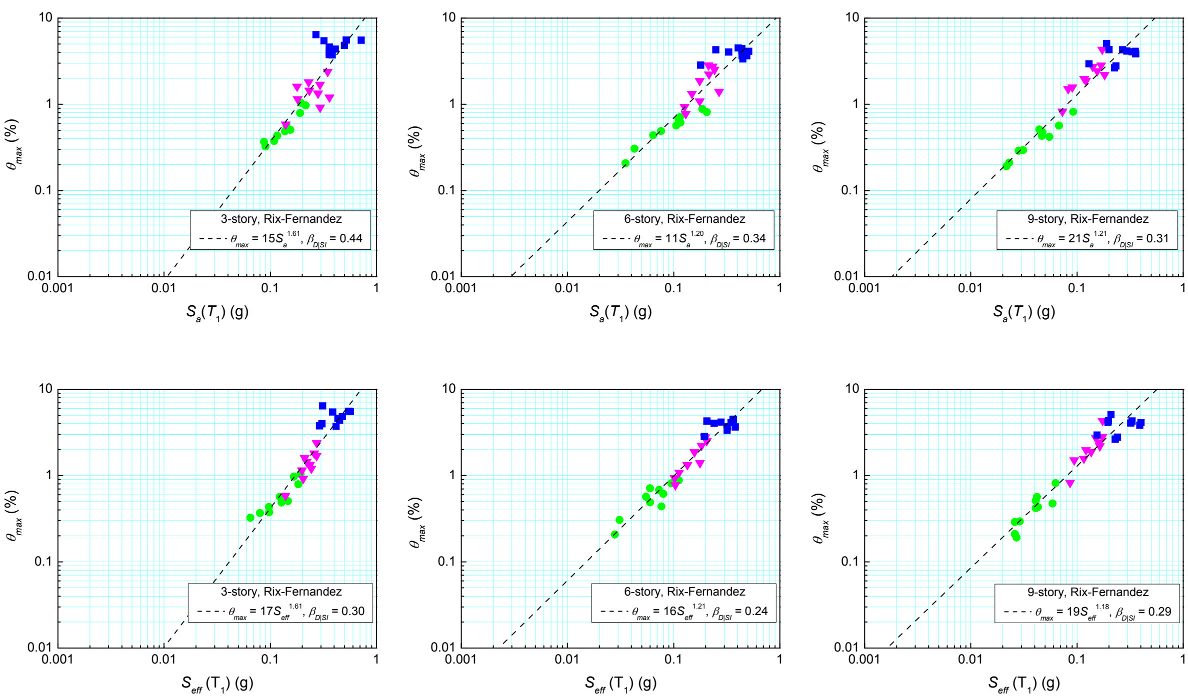


Figure 4. Seismic demands on the frames when the IM = $S_a(T_1)$ and IM = $S_{eff}(T_1)$.

Figure 4 compares the seismic demands on the frames when the IM = $S_a(T_1)$ and IM = $S_{eff}(T_1)$ (*cf.* Eq. 4), and displays the reduction in the scatter in structural responses with the use of $S_{eff}(T_1)$ as an IM. The scatter is represented by $\beta_{D|SI}$ in Eq. 2 and is reduced by 32, 29, and 6% for the three-, six-, and nine-story GLD RC frames, respectively.

Implications for Seismic Risk Assessment — Seismic Fragilities

Seismic fragilities for the three GLD RC frames are derived using the above-mentioned probabilistic demand models in terms of $S_{eff}(T_1)$ (*cf.* Eq. 1). The structural capacities are defined by the limit states (defined in terms of θ_{max}) that correspond to three widely used performance levels (immediate occupancy (IO), life safety (LS), and collapse prevention (CP)) in the

Table 1. Capacity parameters associated with the limit states for each frame.

Parameter	Limit State	3-story	6-story	9-story
	IO	0.2	0.3	0.3
$\hat{C}$ (%)	SD	2	2	2
	CP	5.0	3.9	3.6
	IO	0.25	0.25	0.25
β_C	SD	0.25	0.25	0.25
	CP	0.17	0.08	0.13

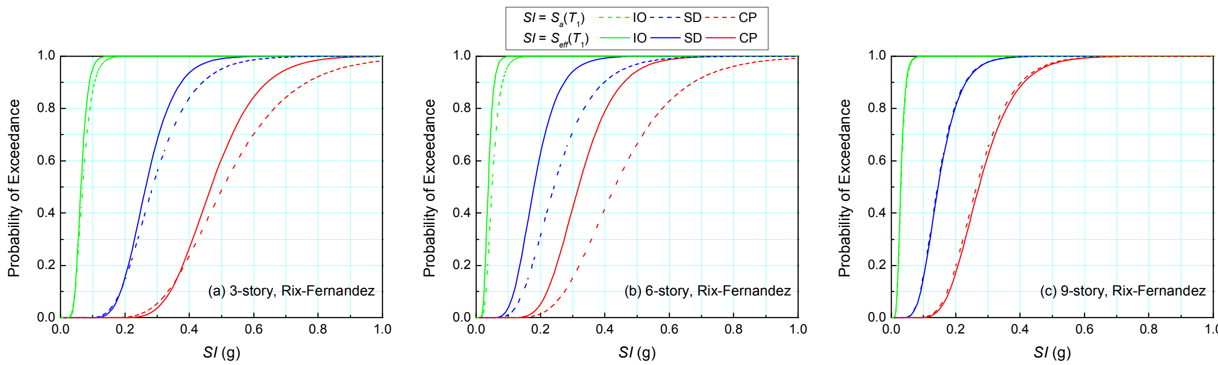


Figure 5. Fragilities in terms of $S_a(T_1)$ and $S_{eff}(T_1)$ for GLD RC frames.

earthquake community (*e.g.*, ASCE 41-06 2007). The IO level is described by the limit below which the structure can be occupied safely without significant repair, and is defined by the value of θ_{max} at which the frame enters the inelastic range. The LS level occurs at a deformation at which "significant" damage has been sustained, but at which a substantial margin remains against incipient collapse. Because this limit is hard to quantify in terms of interstory drift or other structural response parameters, the intermediate level is identified as the interstory drift, θ_{max}, at which significant structural damage (SD) has occurred. Finally, the CP level is defined by the point of incipient collapse of the frame due to either severe degradation in strength of members and connections or significant P-Δ effects resulting from excessive lateral deformations. Table 1 presents the medians and logarithmic standard deviations of θ_{max} associated with these limit states for each GLD RC frame (Celik and Ellingwood 2009, Celik 2007). Finally, the modeling uncertainty, β_M, is assumed to be 0.20, based on the assumption that the modeling process yields an estimate of building frame response that, with 90% confidence, is within ±30% of the actual value (Ellingwood *et al.* 2007).

Figure 5 shows the seismic fragility curves of the frames in terms of $S_{eff}(T_1)$ for the three performance levels — IO, SD, and CP — together with the fragilities derived using the $S_a(T_1)$ as an IM in Celik and Ellingwood (2009). Direct comparisons of these fragilities are not possible as $S_a(T_1)$ is not necessarily equal to $S_{eff}(T_1)$ for the same level of earthquake hazard. However, the comparisons of damage state probabilities that are determined from the fragility curves by entering the fragilities at IM values listed in Table 2 for the 10, 5, and 2% PE in 50 yr median hazard levels (*cf.* Fig. 2), which are illustrated in Fig. 6, indicates that using $S_{eff}(T_1)$ as an IM rather than $S_a(T_1)$ leads to higher collapse probabilities for the three- and six-story GLD RC frames when they are subjected to a maximum considered earthquake ground motion (2% PE in 50 yr earthquake).

Table 2. $S_a(T_1)$ and $S_{eff}(T_1)$ values from the 10, 5, and 2% PE in 50 yr median response spectra of the Rix-Fernandez UHGM for Memphis, TN (Uplands profile).

	$S_a(T_1)$ (g)			$S_{eff}(T_1)$ (g)		
PE in 50 yr (%)	3-story	6-story	9-story	3-story	6-story	9-story
10	0.14	0.09	0.04	0.12	0.06	0.04
5	0.24	0.19	0.12	0.22	0.15	0.13
2	0.51	0.48	0.29	0.52	0.37	0.30

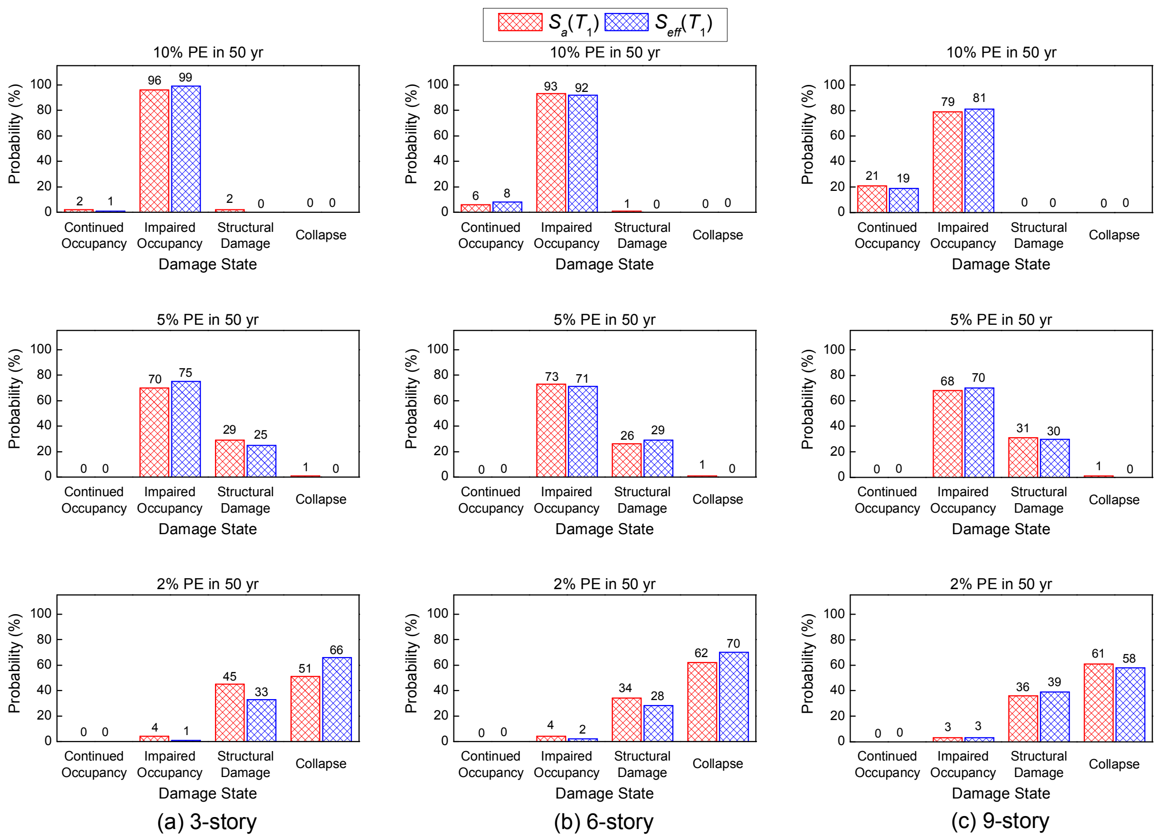


Figure 6. Damage state probabilities from the fragilities in terms of $S_a(T_1)$ and $S_{eff}(T_1)$ for GLD RC frames at 10, 5, and 2% PE in 50 yr earthquake hazard levels for Memphis, TN as stipulated by Rix and Fernandez [2006].

Conclusions

The use of the geometric mean of the spectral accelerations $S_a(T_1)$ and $S_a(1.5T_1)$, *i.e.*, $S_{eff}(T_1)$, as an IM (intensity measure) reduces the scatter in structural responses at a given IM due to record-to-record variability in ground motion ensembles used in nonlinear time history analysis of structures. Collapse damage state probabilities as determined from the fragility curves derived in terms of $S_{eff}(T_1)$ are higher than those determined from the fragilities in terms of $S_a(T_1)$ for the three- and six-story reinforced concrete frames in the Central and Eastern U.S.

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