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EVALUATION OF THE SEISMIC PERFORMANCE OF A CODE-CONFORMING REINFORCED-CONCRETE FRAME BUILDING - PART I: GROUND MOTION SELECTION AND STRUCTURAL COLLAPSE SIMULATION

Christine Goulet¹, Curt B. Haselton², Judith Mitrani-Reiser³, Jonathan P. Stewart⁴, Ertugrul Taciroglu⁵, and Gregory G. Deierlein⁶

ABSTRACT

Emerging performance-based earthquake engineering (PBEE) methods provide a rigorous framework to evaluate the performance of structures, taking into account the significant uncertainties inherent to seismic assessment. This paper and the following companion paper present selected results from a recent study conducted through the Pacific Earthquake Engineering Research (PEER) Center to apply such methods to benchmark the performance of reinforced concrete moment frame buildings designed per current (2003) United States building code provisions. This paper focuses on ground motion selection and structural collapse prediction. We present how Probabilistic Seismic Hazard Analysis results are utilized to characterize the site hazard, specify target spectral acceleration values, and provide pertinent information for ground motion record selection. This paper illustrates that selection based solely on earthquake magnitude and distance may not be sufficient and can lead to conservative predictions of structural performance (e.g., collapse risk). To provide a robust estimate of collapse capacity, the suite of selected ground motions is used in concert with structural analyses that are based on calibrated element models.

Introduction

The results presented in this paper come from a collaborative effort between researchers at the University of California, Los Angeles, Stanford University, and the California Institute of Technology. This effort is referred to as the *Benchmark Project*, which has involved application of a Performance Based Earthquake Engineering (PBEE) methodology to an example structure given currently available tools. The purpose of the Benchmarking project is to predict the seismic performance of a new code-conforming reinforced-concrete (RC) special moment resisting frame (SMRF) building, including the consideration of many relevant uncertain parameters.

PBEE consists of the evaluation, design and construction of structures to meet seismic performance objectives (expressed in terms of repair costs, downtime, casualties, etc.) that are

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defined by stakeholders (owners, society, etc.). The PBEE methodology used for this project was developed by the Pacific Earthquake Engineering Research (PEER) Center (Fig. 1).

This methodology involves conditional probabilities to propagate the uncertainties from each level of the performance analysis, resulting in a probabilistic assessment of the predicted performance.

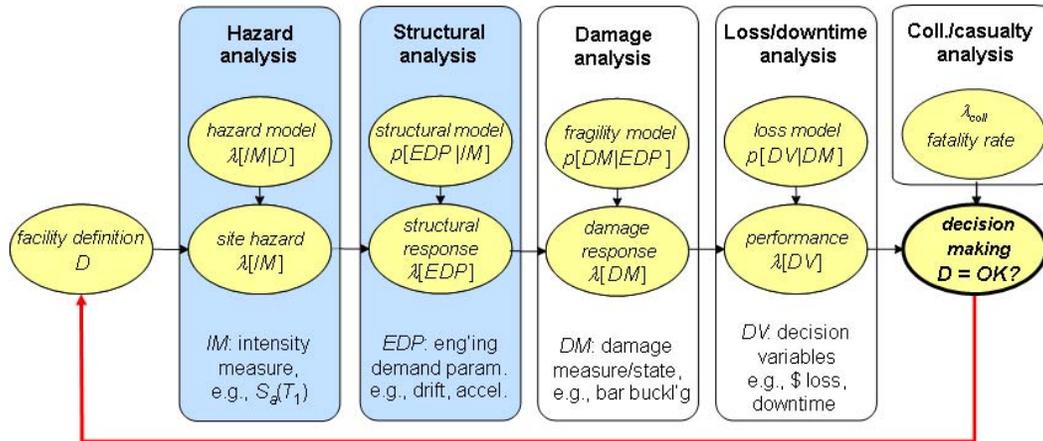


Figure 1. Schematic of the PEER PBEE methodology. Modified from Porter (2003).

For the purpose of this paper, we will focus on the first three steps of the PEER methodology (highlighted on Fig. 1). *Facility definition, D*: the first step of the PEER methodology is the facility definition, site and structure type. *Hazard Analysis*: the site hazard is evaluated and expressed in terms of an intensity measure (IM), given D . Ground motion records representative of the site hazard are selected. *Structural Analysis*: a detailed model of the structure accounting for element non-linearity is created; the ground motion records are used to simulate plausible seismic demand and the structural response is measured in terms of engineering demand parameters (EDP), such as maximum floor acceleration or inter-story drift. The damage analysis and the loss modeling steps are beyond the scope of this paper but are presented in the companion paper (Mitrani-Reiser et al. 2006)

Site Selection and Description

For the benchmark project, the goal was to locate the building on typical site conditions for an urban region within California. We wanted to select a site for which the ground motions are not dominated by unusual features specific to the region (e.g. basin edge effects). A number of other factors were also considered and are presented in more detail in Haselton et al. (2005). The selected site is the LA Bulk Mail facility in Bell, CA (coordinates 33.996° Lat. -118.162° Long.), located south of downtown Los Angeles. One would expect some effect of near-source ground motions, since the site is within 20 km of 7 faults. However, no single major fault produces near-fault motions that dominate the site hazard, and the contributions of near fault motions from the other faults are actually very typical of the Los Angeles area. This site met our selection criteria and also had the advantage of available high quality geotechnical data from the ROSRINE program (<http://geoinfo.usc.edu/rosrine>, last accessed August 2005). The site is located on deep sediments, mostly Quaternary alluvial deposits, near the middle of the Los Angeles Basin. The upper 30 m consist of sands and silts with traces of clay and cobbles with a corresponding average shear wave velocity $V_{s-30} = 285$ m/s (NEHRP soil category D).

Site Hazard Characterization

One of the main objectives of the benchmark project is to consider and quantify uncertainties at each level of the PBEE methodology, from *IM* to *DV*. Within this context, the purpose of the ground motion hazard characterization is two-fold. First, we performed a Probabilistic Seismic Hazard Analysis (PSHA) to characterize *IMs* for seven hazard levels. The *IM* selected for this project is the 5% damped spectral acceleration at the building's first mode period $S_a(T_I=1.0\text{ s})$. The PSHA considered several sources of epistemic uncertainty, including alternative ground motion prediction equations and alternative estimates of fault slip rate. Second, we compiled suites of ground motion records that are compatible with the selected hazard levels for use in structural response analyses. Mean uniform hazard spectra (sampling across the sources of epistemic uncertainty) are plotted in Fig. 2 for the seven hazard levels selected. From these results, we extracted the target $S_a(T_I)$ values to be matched when scaling the ground motion records.

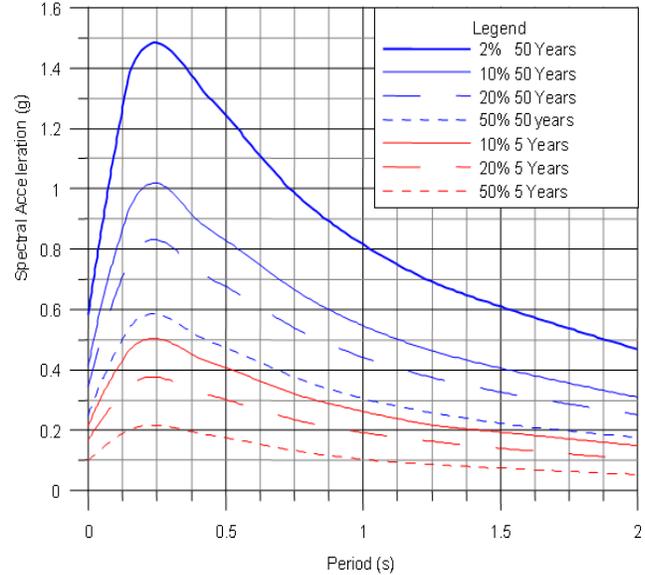


Figure 2. Mean uniform hazard spectra (5% damping) for the LA Bulk Mail site for seven hazard levels

Record Selection Methodology

Records were selected from a database of 1495 3-channel recordings (two horizontal and one vertical component). However, since records with rock site conditions or poorly defined estimates of magnitude and distance were not considered, the effective database size for record selection was reduced to 789, combining records from 41 earthquakes with $M > 5$. To capture the variability of future ground motions at the site, 20 to 30 records were selected whenever possible for each hazard level. Our objective was to select records that are compatible with the results of hazard analysis and that are appropriate for the site location. Identification of the most critical source conditions is accomplished through disaggregation of the seismic hazard (e.g., Bazzurro and Cornell, 1999), which separates the contributions from different magnitudes and site-source distances. The selected records have similar magnitude, site-source distance, causative fault focal mechanism, and site-source azimuths (for near-fault effects) to “target” values (as defined by the PSHA results).

A disaggregation scheme was used that groups the relative contribution of earthquakes into bins of magnitude (M), distance (r), and epsilon (ε). Baker and Cornell (2005) proposed the parameter ε (“epsilon”), which is a period-dependent quantity that indirectly characterizes spectral shape. Epsilon is a property of the ground motion record defined by:

$$\varepsilon = \frac{\ln(S_a)_{data} - \ln(\mu_{S_a})}{\sigma_{S_a}}$$

where μ_{Sa} and σ_{Sa} are predicted values of the median and standard deviation (in ln units) of S_a from an attenuation relationship (Abrahamson and Silva (1997) was used for this project). All of the above quantities are evaluated at the period of interest; in this case we used the spectral acceleration of the first mode, $S_a(T_1=1.0\text{ s})$. The physical interpretation of ε is the offset (in number of standard deviations) between the value of the record's IM and the expected value from an attenuation relationship.

Baker and Cornell found ε to be an effective indicator of whether a particular record will cause a relatively large or relatively modest nonlinear response (as quantified by EDP response). The hazard disaggregation based on M , r and ε is shown in Fig. 3 for two hazard levels.

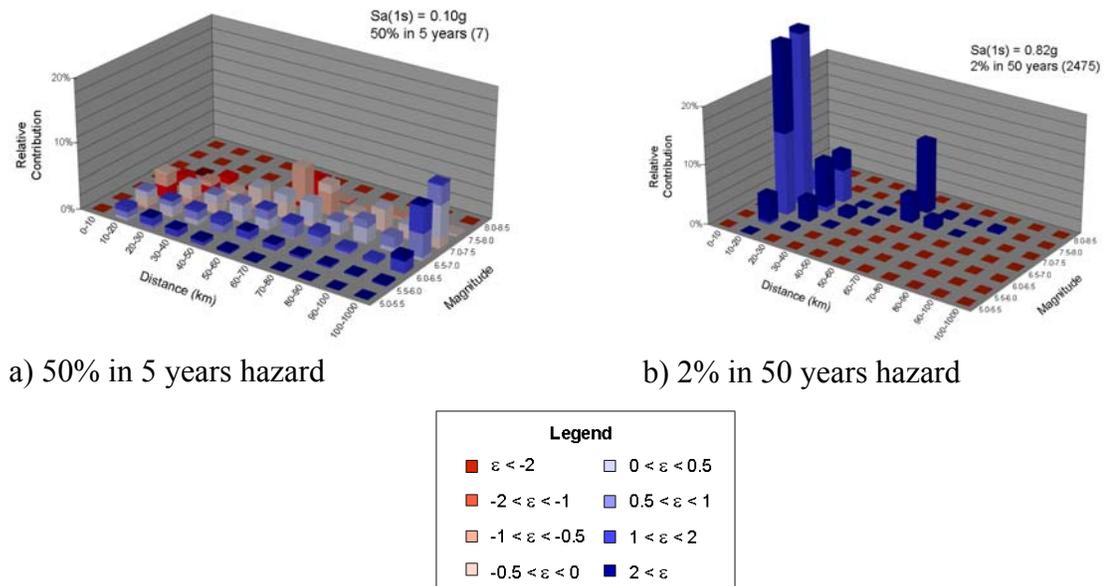


Figure 3. Disaggregation of hazard for two hazard level at $T=1.0\text{s}$.

Fig. 3 shows that at low hazard levels, many magnitude and distance bins contribute similarly to the estimated ground motion. For low hazard levels (Fig. 3a), nearby sources with higher magnitudes are characterized by relatively low, often negative, ε values. This is because large magnitude earthquakes on nearby faults are likely to exceed a low ground motion level, thus producing negative ε . Sources at larger distances with lower magnitudes are associated with larger ε values because the ground motions from these events would need to be significantly larger than average to exceed even a low ground motion level. As hazard level increases (Fig. 3b), particular magnitude and distance ranges begin to dominate. This “specialization” of sources can be explained by only particular sources (M and r combinations) generating the large earthquakes necessary to produce severe ground motions. This is also why ε tends to be positive for high hazard level events.

Records were selected independently for each hazard level based on the geometric mean of the two horizontal components, which is consistent with the ground motion prediction equations used in PSHA. We therefore end up with two horizontal components which are used for the dynamic structural simulations. For each hazard level, records were extracted that match the target M , r , and ε based on disaggregation results and other considerations (Haselton et al.

2005). All the selected records were scaled to their hazard-specific target $\{S_a(T_1)\}_{target}$. In this paper, the term “scaling” refers to this intrabin matching of the target $IM, S_a(T_1)$.

Benchmark Building Design and Structural Model

The benchmark building is a four-story office building designed according to the 2003 International Building Code (ICC 2003). For the full benchmark effort, eight designs were considered; this paper focuses on a single perimeter frame design (Design A, Fig. 4) that was designed to have expected levels of overstrength (125% of Code required strength, etc.). The full report on this research has more detailed information regarding the building designs (Haselton et al., 2005).

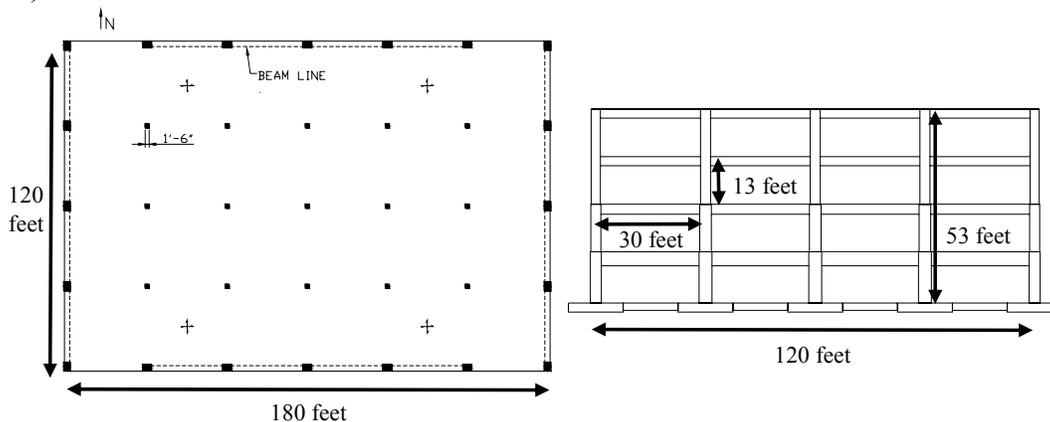


Figure 4. Plan and elevation of four-story office building, Design A.

We created a two-dimensional model of a typical four-bay frame in the N-S direction. This model consists of finite joint elements, column base springs, and lumped plasticity beam-column elements. The lumped plasticity element model was developed with the expressed purpose of capturing the important modes of deterioration that precipitate sideways collapse (Ibarra et al. 2005; Ibarra 2003). The model captures cyclic strength deterioration and negative post-failure slope, which is critical for realistic collapse analysis. This model has been calibrated to test data (e.g., Panagiotakos and Fardis 2001; Haselton et al. 2005; PEER 2005). We did not consider the effects of the gravity system or non-structural elements (partitions, etc.) for the analyses presented in this paper. We have used a software developed by the PEER center (OpenSees 2005) for the structural modeling and analysis.

Collapse Simulation and Collapse Capacity Estimates: Recommended Approaches for Considering Ground Motions

Incremental dynamic analysis (IDA, Vamvatsikos 2002) was used to simulate sideways collapse, which occurs as the point of dynamic instability when story drift increases without bounds. The period used for ground motion scaling is 1.0 second, but the fundamental period of the structural model for Design A is closer to 1.3 seconds⁷. Fig. 5 shows the IDA results for 34 ground motion records selected considering ϵ for the 2% in 50 year ground motion level. Fig. 5a

⁷ The 1.0 second period was used for because it is the average period for all eight designs of this Benchmark study (periods range from 0.8 sec. to 1.3 sec.). The difference between fundamental period and the period used for scaling will cause ϵ to be more important; even so the impacts of ϵ shown in this paper are consistent with other research.

shows the results obtained from all 68 ground motion components (two components per record). Fig. 5b shows the results obtained by using the horizontal component of each record that causes collapse at a lower Sa level. The method illustrated in 5b is used in this study to approximately capture the fact that ground motions are three-dimensional in nature and the building collapses when it collapses in *either* of the two horizontal directions.⁸ This is a simple, yet important, concept that can significantly impact the collapse capacity estimates. Comparison of Figs. 5a and 5b clearly show the differences in the collapse capacity estimates resulting from the two methods.

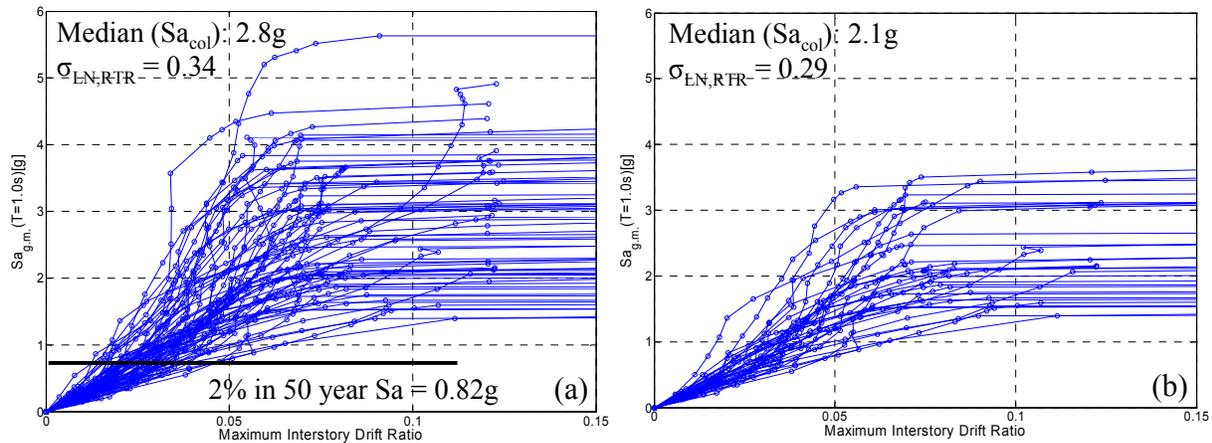


Figure 5. Incremental dynamic analysis for Design A, using a) both horizontal components of ground motion, and b) the horizontal component that first causes collapse.

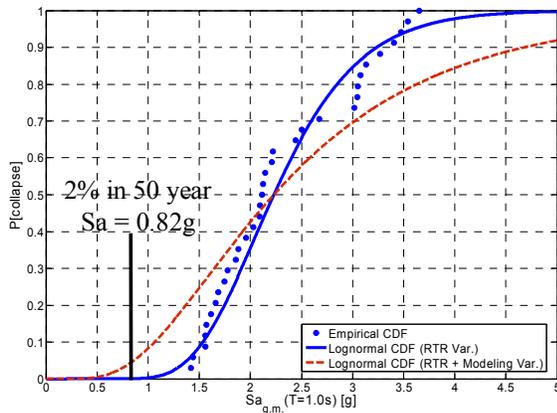


Figure 6. Empirical collapse capacity CDF and lognormal collapse CDFs including and excluding modeling and

record-to-record variability. Fig. 6 shows that the probability of collapse for the 2% in 50 year ground motion is < 1% with only record-to-record variability and 4% when modeling uncertainties are included. This shows that modeling uncertainties significantly impact collapse probabilities. We observed similar effects on collapse rates.

Fig. 6 follows directly from Fig. 5b, showing a blue dot for the collapse capacities predicted for each of the 34 ground motion records. The blue solid line shows the lognormal cumulative density function (CDF) fit to these 34 collapse capacities, and illustrates the effect of record-to-record variability. The red dashed line incorporates the additional variability in the collapse capacity estimate that comes from modeling uncertainties such as uncertainties in plastic rotation capacity, strength, and stiffness ($\sigma_{LN,modeling} = 0.50$; Haselton et al. 2005, chp. 6). Modeling uncertainty is incorporated using the square-root-of-the-sum-of-the-squares of $\sigma_{LN,RTR}$ and $\sigma_{LN,modeling}$, to find the new expanded σ_{LN} of

⁸ This approximate method considers only the differences between the two horizontal components of ground motion, not any 3D *structural* interactions (we do not expect these effects to be significant for this building).

Table 1 presents the collapse predictions using the approach presented in this paper. This recommended approach: a) involves a selection of ground motions considering ϵ , b) approximately considers the 3D ground motion effects, and c) utilizes a well-calibrated structural model that captures the important modes of deterioration that precipitate sidesway collapse. The table includes the counted median collapse capacity, the parameters for the fitted lognormal distribution, and estimates of collapse probability and mean annual frequency of collapse⁹ ($\lambda_{\text{collapse}}$). $\lambda_{\text{collapse}}$ is computed by numerically integrating the collapse CDF with the hazard curve (Ibarra 2003, Eq. 7.10). Table 1 shows that the probability of collapse is quite low (4%) for the 2% in 50 years (the equivalent yearly rate of collapse is similarly low at 65×10^{-6}).

Table 1. Summary of estimates of collapse capacity, probability, and rate.

			With only record-to-record variability (lognormal fit)			With record-to-record and modeling variability* (lognormal fit)		
Counted Median (Sa,col)	Collapse Margin (median / Sa _{2/50})**	$\mu_{LN}(Sa,col)$	$\sigma_{LN}(Sa,col)$	$\lambda_{\text{collapse}}$ (10 ⁻⁶)	P[Col Sa _{2/50}]**	$\sigma_{LN}(Sa,col)$	$\lambda_{\text{collapse}}$ (10 ⁻⁶)	P[Col Sa _{2/50}]**
2.1	2.6	0.81	0.29	6.5	0.00	0.58	65	0.04

*All for sigmaLn modeling = 0.50

** 2% in 50 year ground motion level: Sa(1sec) = 0.82g

Collapse Capacity Estimates: Sensitivities to Ground Motion Selection with ϵ and Approximate 3D Method

The recommended methods proposed above not often employed in engineering practice and research. Therefore, this section illustrates the impacts that that ϵ -based ground motion selection and approximate 3D considerations have on the collapse predictions. This section compares collapse capacity CDFs obtained using each of the methods and then closes by comparing the collapse probabilities and rates for the recommended method and the methods that do not consider ϵ or three-dimensional ground motion effects.

Effects of ϵ on Predicted Collapse Capacity

To illustrate the effects that ϵ -based ground motion selection has on collapse capacity predictions, we selected an additional set of 30 ground motions that does not consider ϵ but is still roughly consistent with motions that may cause collapse of new buildings: $M > 6.5$, $R > 10\text{km}$, and $\text{PGA} > 0.2\text{g}$ (Kircher 2005; PEER 2005).

Fig. 7a shows the median of the spectra of each ground motion set scaled to the 2% in 50 year target spectral value ($Sa_{2/50}=0.82\text{g}$) at 1.0 second. The ϵ -selected set contains 34 ground motions and the set selected without considering ϵ contains 30 motions. This figure shows that the median spectrum of the ϵ -selected set is significantly lower than the non ϵ -selected set at extended periods. For these record sets, at periods much greater than 1.0 seconds, the median spectrum of the ϵ -selected set is almost one standard deviation below the median of the non ϵ -selected set (these observation not expected to be general for all record sets). Nonlinearity in structural response near collapse will cause an increase in the “effective period” of the structure, so this difference in spectral shape will translate into differences in collapse capacity predictions.

⁹ This is the *mean estimate* of the mean annual frequency of collapse

Haselton and Baker (2006) show that, for ductile structures such as this, the “effective period” at collapse is approximately twice the fundamental first mode period.

Fig. 7b shows the collapse capacity prediction (empirical collapse CDFs) obtained by using each of the two ground motion sets. This shows that the median collapse capacity differs by 20% for the two ground motion sets and the $P[\text{collapse}|S_a]$ can change by up to 30% (amount of change depends on S_a level). For example, for $S_a = 0.5g$ the $P[\text{collapse}|S_a] = 0\%$ for both ground motion sets; while at $S_a = 1.4g$ the $P[\text{collapse}|S_a]$ is 0% for the ϵ -selected ground motion set and 30% for the set selected without considering ϵ . This 30% difference is typical of most S_a levels between 1.4g and 3.0g.

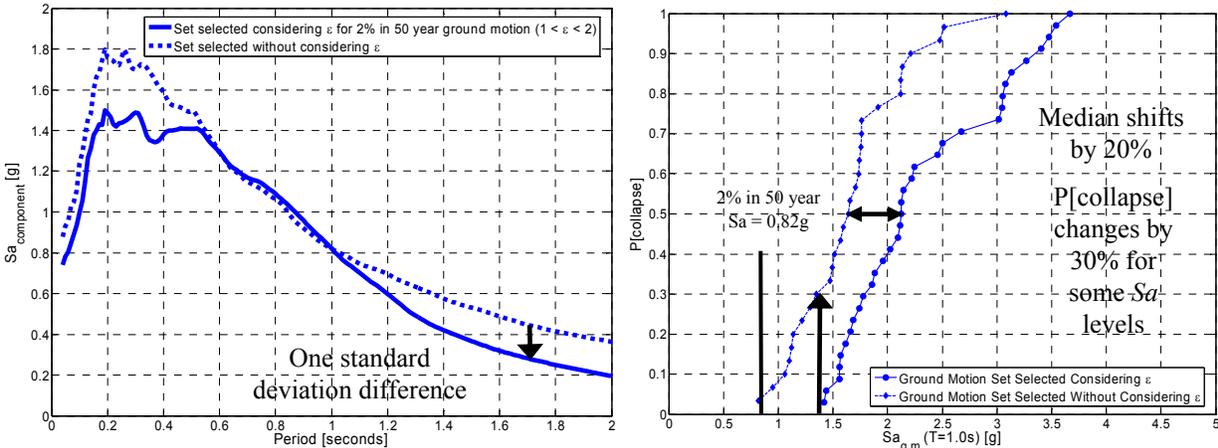


Figure 7. a) Comparison of median spectra for two ground motion record sets, b) comparison of collapse capacity CDFs using two ground motion records sets

Impacts of 3D Effects on Predicted Collapse Capacity

To judge the impact on collapse capacity of considering 3D ground motion effects, Fig. 8 shows the collapse predictions from Fig. 5a-b in CDF format. As compared to Fig. 7b, this shows similar changes in the median collapse capacity and probability of collapse (at higher levels of S_a). However, one important difference is that the tail of the collapse CDF is affected differently (compare Figs. 7a and 8). This shows that for ground motion levels near the tail of the distribution ($S_a < 1.5$ in this case), considering 3D effects is less important than considering the effects of ϵ . However, for ground motions near the median of the distribution ($S_a=2.0-2.5g$ in this case), the three-dimensional effects and ϵ effects are both of similar importance and can change the predicted probability of collapse by 20-30%.

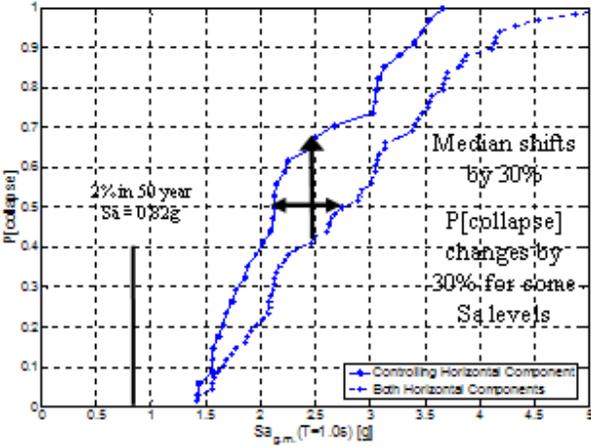


Figure 8. Collapse capacity CDFs with and without considering 3D ground motion effects

This suggests that for newly designed buildings (where the S_a values of interest, e.g. $S_{a2/50}$, are typically near the tail of the collapse capacity distribution) three-dimensional effects can be *less* important in magnitude, whereas for existing fragile buildings (where the S_a values of interest can be near the median of the collapse capacity distribution) considering the three-dimensional ground motion effects is more critical.

Comparisons of Collapse Probability and Rate

Table 2 presents the collapse probability and rate for each of the three methods: a) the recommended method presented in this paper, b) the method not considering ϵ in ground motion selection, and c) the method not considering 3D ground motion effects. To avoid repetition, a summary of results of Table 2 is left for the conclusion of this paper.

Table 2. Summary of estimates of collapse capacity, probability, and rate.

Ground Motion Set	Number of Ground Motions	Consideration of Horizontal GM Components	Counted Median ($S_{a,col}$)	Collapse Margin (median / $S_{a2/50}$)**	$\mu_{LN}(S_{a,col})$	With only record-to-record variability (lognormal fit)			With record-to-record and modeling variability* (lognormal fit)		
						$\sigma_{LN}(S_{a,col})$	$\lambda_{collapse} (10^{-6})$	P[Col $S_{a2/50}$]**	$\sigma_{LN}(S_{a,col})$	$\lambda_{collapse} (10^{-6})$	P[Col $S_{a2/50}$]**
Considering ϵ	34 (x2 horiz. comp.)	Controlling	2.1	2.6	0.81	0.29	6.5	0.00	0.58	65	0.04
Not Considering ϵ	30 (x2 horiz. comp.)	Controlling	1.7	2.1	0.47	0.30	45	0.01	0.58	330	0.13
Considering ϵ	34 (x2 horiz. comp.)	All	2.8	3.4	0.99	0.34	3.5	0.00	0.60	52	0.02

*All for sigmaLn modeling = 0.50

** 2% in 50 year ground motion level: $S_a(1sec) = 0.82g$

Conclusions

For the specific case of the benchmark structure, we conclude that:

- Selecting ground motion sets considering ϵ increased the median predicted collapse capacity by 20%, decreased the P[collapse | $S_{a2/50}$] by 10%, and decreased the annual rate of collapse by a *factor of five-seven*. Similar research regarding ϵ (Haselton and Baker 2006) shows that change to the annual rate of collapse is more generally *three-six*.
- Considering three-dimensional ground motion effects decreased the median predicted collapse capacity by 30%, increased the P[collapse | $S_{a2/50}$] by 2%, and only slightly increased the annual rate of collapse. The small changes in collapse probability and rate come from this structure being highly ductile and having small probabilities of collapse at both $S_{a2/50}$ and at the S_a with high rates of occurrence.
- ϵ -selection and three-dimensional ground motion effects change the median collapse capacity by a similar amount. However, the shapes of the collapse capacity distributions are changed much differently. Consequently, for S_a levels near the lower-tail of the distribution, ϵ has a much greater impact.

We have illustrated that collapse predictions are sensitive to ϵ , a ground motion parameter that considers the spectral shape around the spectral period of interest. We have also shown that performing analyses using both horizontal components of ground motions allows the engineer to select the component that is most-likely to initiate collapse, therefore considering what we have called the *three-dimensional effect* of ground motions. The effects of both ϵ and three-dimensional ground motion considerations *depend drastically* on the hazard curve and on the location of the collapse capacity distribution relative to $S_{a2/50}$. More specifically, depending

on the S_a value, $P[\text{collapse}|S_a]$ can vary from 0 to +/-30%. Finally, we have observed from our simulations that the annual rate of collapse is a more stable indicator of collapse risk, especially when probabilities of collapse are small. Table 2 shows that the $P[\text{collapse}|S_{a_{2/50}}]$ vary from 0-1%, while the annual rate of collapse varies from $3 \cdot 10^{-6}$ - $45 \cdot 10^{-6}$. Not considering the annual rate of collapse may lead to biased conclusions.

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