Assessing the Seismic Collapse Risk of Reinforced Concrete Frame Structures, Including the Effects of Modeling Uncertainties

Abbie Liel
Ph.D. Candidate, Stanford University
Blume Earthquake Engineering Center; Stanford, CA 94305

Curt Haselton
Assistant Professor, California State University, Chico
Department of Civil Engineering; Chico, CA 95929

Gregory Deierlein
Professor, Stanford University
Blume Earthquake Engineering Center; Stanford, CA 94305

Jack Baker
Assistant Professor, Stanford University
Terman Engineering Center; Stanford, CA 94305

Abstract

A primary goal of seismic provisions in building codes and retrofit legislation is to protect life safety through prevention of structural collapse. To evaluate the extent to which these specifications meet this objective, the authors have conducted detailed assessments of the collapse performance of both modern reinforced concrete (RC) special moment frames (SMF) and existing RC non-ductile moment frames.

Many aspects of the assessment process, including the treatment of modeling uncertainties, can have a significant impact on the evaluated collapse performance. Approaches for evaluating the effects of modeling uncertainties are described in this study. Uncertainties in strength, stiffness, deformation capacity, and cyclic deterioration are considered for ductile frame structures of varying heights. Due to the computationally intensive nature of these analyses, the effect of these modeling uncertainties is assessed through creation of a response surface from the results of sensitivity analyses. From the response surface, Monte Carlo simulation is used to quantify the impact of these uncertainties on the predicted collapse capacity of each structure.

1. Introduction

The process of assessing structural seismic performance at the collapse limit state through nonlinear simulation is highly uncertain. For assessment of an individual building design, there is significant uncertainty in the future ground motion that may occur, both in terms of the intensity (given by the site specific hazard curve), and the frequency content and other characteristics of the ground motion (termed record-to-record variabilities). Similarly, there are uncertainties in the structural modeling process and the extent to which the idealized model accurately represents real behaviour. Firstly, there may be several options of what type of model to use. Once a particular model is chosen, the modeling

parameters used in the structural model are again a source of uncertainty, as the actual strength and deformation may differ from the expected values. These uncertainties are referred to as modeling uncertainties. Additionally, if the assessment is based on a possible future design, there is also design uncertainty, which accounts for variability in engineering design choices, given the prescriptive code requirements that govern design. Other sources of uncertainty, including human error and construction quality, are not considered in this study.

These sources of uncertainty are critical components of the probabilistic assessment of a structure's collapse capacity. Record-to-record variabilities are directly incorporated into the analysis procedure through use of a sufficiently large set of ground motion records. The problem considered here is how to realistically and expediently quantify the effects of modeling uncertainties. Many researchers have varied uncertain modeling parameters, including damping, mass, and material strengths, and concluded that these variations make a relatively small contribution to the overall uncertainty in seismic performance predictions. However, these studies have focused primarily on pre-collapse performance. In contrast, we show that the modeling uncertainties associated with deformation capacity and other parameters critical to collapse prediction have a significant effect on the assessed collapse performance.

To begin, we provide an overview of the collapse assessment procedure and results for a set of structures: RC moment frames in high seismic regions. We then review methods for quantifying the effects of uncertainty in element and system level modeling, and propose a procedure that combines response surface analysis and Monte Carlo simulation. This procedure is applied to three RC SMFs of varying heights. Finally, we compare the results obtained in this study with first-order second-moment reliability methods, which are easier to implement, but rely on the validity of key, simplifying assumptions. We focus primarily on the effects of modeling uncertainties on the spectral acceleration at collapse, but other measures such as the peak interstory drift ratio at collapse could also be explored.

2. Overview of Collapse Assessment Procedure and Results

This study is primarily concerned with assessing structural collapse due to earthquakes, focusing in particular on RC frame structures. The procedure used for collapse assessment utilizes the performance-based earthquake engineering methodology developed by the Pacific Earthquake Engineering Research center, which provides a probabilistic framework for relating ground motion intensity to the structural response through structural simulation (Deierlein 2004).

Simulation of global sidesway collapse uses the Incremental Dynamic Analysis (IDA) technique (Vamvatsikos and Cornell 2002). In IDA, the analytical model of a structure is subjected to a ground motion record, and the structural response is simulated. This analysis is repeated, each time increasing the scale factor on the ground motion's intensity, until that record causes structural collapse in a sidesway mode. This process is then repeated for an entire suite of ground motion records, to capture the record-to-record uncertainty in the response.[1] In these analyses, the ground motion intensity measure is the spectral acceleration at the first mode period of the building [$Sa(T_1)$]. The outcome of the IDA procedure is an empirically obtained cumulative probability distribution relating probability of collapse to the $Sa(T_1)$ of the ground motion. When there are possible failure modes that are not captured in the simulation model, these can be incorporated through post-processing, combining

---

[1] For this study, the ground motions were selected to represent large earthquakes with moderate fault-rupture distances (i.e., non near-field conditions). This is the basic Far-Field ground motion set selected by Haselton and Kircher as part of an Applied Technology Council project, ATC-63. These records were selected without consideration of epsilon, a measure of spectral shape which has been shown to have a significant impact on collapse capacity (Haselton 2006).

component or system fragility curves with the IDA results (see Liel et al. (2006)). Several different metrics can be used to quantify collapse performance: collapse capacity margin (the ratio of median collapse capacity to the maximum considered earthquake (MCE) demand), probability of collapse conditioned on the MCE (or other hazard level of interest), and mean annual frequency of collapse (obtained by integrating the collapse probability distribution with the hazard curve for a particular site).

This procedure was used to assess the performance of both modern and existing RC frame buildings. The nonlinear analysis model for each structure consists of a 2-D three-bay frame created in OpenSees, as shown in Figure 1. The models capture material nonlinearities in beams, columns, and beam-to-column joints, along with P-Delta effects. The beam-column hinges are modeled using the backbone shown in Figure 1b and the associated hysteretic rules; the properties of these hinges are obtained from systematic calibration to 255 experimental tests, as described in Haselton et al. (2007). The joints are modeled as finite size with a joint shear spring. Mean values are utilized for all modeling parameters, in order to represent the expected behavior.

Haselton (2006) studied 30 code-conforming RC SMFs of varying height (1 - 20 stories), and evaluated the collapse capacity of each structure.[2] These structures are assumed to be at a specified site in Los Angeles in the transition zone, for which the hazard curve has been defined through probabilistic seismic hazard analysis (Goulet et al. 2007). From the analysis, the collapse margins (relative to the MCE) range from 1.1 to 2.1 for this set of structures. The collapse probabilities conditioned on the MCE ground motion vary from 0.12 to 0.47. The mean annual frequency of collapse ($\lambda_{collapse}$) ranges from 2.2×10^{-4} to 25.5×10^{-4} collapses/year, corresponding to a collapse return period of between 400 and 4500 years.[3],[4] These collapse assessments reveal that the collapse performance is relatively stable for structures of different heights, and that perimeter frames typically have worse performance than space frames (because of the higher inherent overstrength in space frame design and greater dominance of P-Δ effects in perimeter frames). A similarly comprehensive study of non-ductile RC moment frames, of the type constructed in the 1960s and 1970s in California, is underway. In general, it is shown that these structures are considerably more likely to collapse in earthquakes due to the lack of capacity design requirements and detailing provisions (Liel et al. 2006).

3. Treatment of Modeling Uncertainties

3.1. Review of Previous Research

Recognizing the uncertainties in the structural modeling process, a variety of approaches have been used to study the effects of these uncertainties on the resulting structural response and performance predictions, such as those described above for reinforced concrete frame structures.

Sensitivity analysis provides a simple method for computing the effects of modeling uncertainties on response quantities of interest. The effect of each random variable on structural response is determined by varying a single modeling parameter and re-evaluating the structure's performance. These studies,

---

[2] These structures are fully designed according to the provisions of ASCE 7-02, ACI 318-02 and IBC 2003.

[3] The values reported here include the effects of modeling uncertainties obtained through the FOSM procedure and the mean estimates approach (discussed later), where it is assumed $\sigma_{ln,modeling} = 0.45$. The $\sigma_{ln,modeling} = 0.45$ value was obtained through a detailed study for a 4-story reinforced concrete building, and it was assumed that this value is appropriate for the other structures studied. When structural modeling uncertainties are excluded from the analyses, $\lambda_{collapse}$ decreases to 0.3×10^{-4} to 8.3×10^{-4} collapses/year.

[4] As noted previously, these collapse assessments are conservative, because they do not include an adjustment for spectral shape.

eg. those conducted by Esteva and Ruiz (1989), Porter et al. (2002), Ibarra (2003), or Aslani (2005), are used to identify those modeling parameters that have the most significant impact on the response.

First-order-second-moment (FOSM) reliability approaches provide a simple method for propagating modeling uncertainties to quantify their effect on structural response. In FOSM, the variance of the response due to various sources of uncertainty is computed by assuming the limit state function is linear. The needed gradients of the linearized limit state function can be obtained through perturbation of individual random variables in a series of sensitivity analyses. Unfortunately, the analyses may become inaccurate for highly nonlinear functions. In addition, the FOSM method uses only information about the first and second moments of the input random variables (mean and variance), and is inappropriate for problems in which the modeling uncertainties may alter the prediction of the median as well as the dispersion (Baker and Cornell 2007).

Several researchers have explored the effects of modeling uncertainties with FOSM, including Ibarra (2003), Lee and Mosalam (2005). Haselton studied the effects of modeling uncertainties on the collapse capacity of a code-conforming 4-story RC SMF designed for a high seismic region in California (Haselton 2006; Haselton et al. 2006). The authors used a finite difference approach to compute the sensitivity to each random variable for use in FOSM input. When partial correlation assumptions were used, the most realistic case, the logarithmic standard deviation contribution from modeling and design uncertainties on collapse capacity is 0.45 (or roughly equivalent to the record-to-record variability). This work by Haselton et al. provides the basis for comparison for this study.

An alternative approach uses Monte Carlo simulations to determine the effect of modeling uncertainties on the structural response predictions. The Monte Carlo procedure generates realizations of each random variable, which are inputted into a simulation model, and the model is then analyzed to determine the collapse capacity. When the process is repeated for thousands of sets of realizations a distribution on collapse capacity results associated with the input random variables is obtained. The simplest sampling technique is based on random sampling using the distributions defined for the input random variables, though other techniques, known as variance reduction, can decrease the number of simulations needed. These Monte Carlo procedures can become computationally very intensive if the time required to evaluate each simulation is non-negligible (Helton and Davis 2001; Rubinstein 1981).

The computational effort associated with full Monte Carlo simulation can be reduced through combination with response surface analysis. A response surface is a simplified functional relationship or mapping. As such, it can be used to approximate a limit state function as a function of selected input random variables. The price of this efficiency is a loss of accuracy in the estimate of the collapse capacity, which depends on the degree to which the highly nonlinear predictions of structural response can be accurately represented by the simplified response surface (Helton and Davis 2001; Pinto et al. 2005). Ibarra (2003) analyzed the collapse capacity of a single degree-of-freedom system and used a response surfaced to represent the collapse capacity as a function of post-capping stiffness. Ibarra's study found, for that particular case, that the simplified FOSM procedure, the full Monte Carlo procedure, and the combined response surface/Monte Carlo approach produced comparable result.

Whichever procedure is used, correlations between the input random variables under consideration can determine how significantly the modeling uncertainties impact the structural response (Haselton 2006; Val et al. 1997). Possible correlations include both correlations between the properties of a particular element, and correlations over the height of the building. There is insufficient data to quantify these correlations, so expert judgment is typically used. In general, increased correlation tends to increase the dispersion in the response quantity of interest, which generally leads to decreased collapse performance; the fully correlated case is typically considered to be conservative.

Once the effects of modeling uncertainties have been predicted there is still significant debate related to interpretation of these results, centering on how the effects of modeling uncertainties should be combined with the effects of other sources of uncertainty, such as record-to-record variabilities. For this purpose, different sources of uncertainty are sometimes characterized as either "aleatory" (randomness) or "epistemic" (lack of knowledge).

One common approach for combining the effects of different sources of uncertainty is the confidence interval approach, through which we can make statements about structural response fragilities at a specified level of confidence (Cornell et al. 2002). The confidence interval method is illustrated in the collapse fragilities shown in Figure 2a. Record-to-record variability (treated as aleatory) is shown by the cumulative distribution function obtained directly from IDA analyses (blue), and the epistemic uncertainty (related to modeling variability) creates the distribution on the mean(green). The distribution associated with epistemic uncertainty may be obtained from FOSM, Monte Carlo simulations, or expert judgment. In order to make predictions at a specified confidence level, the cumulative aleatory distribution is shifted to the appropriate percentile on the epistemic distribution. Thus, the probabilities associated with the shifted distribution in Figure 2a (red) are consistent with a 90% prediction of confidence, accounting for both aleatory and epistemic sources of uncertainty. Although this approach is conceptually appealing, the resulting structural performance predictions become highly dependent on the level of confidence chosen. In addition, it requires distinguishing between aleatory and epistemic uncertainties, which can quickly become a philosophical debate.

A second approach, referred to as the mean estimates approach, can be used to combine the contributions of the epistemic and aleatory uncertainties in structural response fragilities. Two assumptions are needed: first, it is assumed that the two distributions are independent and, second, that both can be well-described using lognormal distributions such that the epistemic uncertainty is represented by a lognormal distribution on the median collapse capacity (Cornell et al. 2002). If these two conditions are satisfied, the logarithmic standard deviations associated with each can be combined using the square-root of the sum-of-the-squares approach (SRSS) to obtain the total variance associated with the fragility. When the mean estimates approach is used, the median is unchanged when modeling uncertainties are incorporated, but the variance increases, as shown in Figure 2b. This approach does not distinguish between aleatory and epistemic uncertainties.

A third approach is similar to the mean estimates approach, except that it does not rely on the assumption of independence, and can be used to quantify the effects of modeling uncertainty on both the mean and variance of the structural response fragility. By viewing the results of Monte Carlo simulations as alternate potential descriptions of reality, we interpret both the modeling and record-to-record uncertainties as leading to uncertainties in the probabilities defining the collapse fragility curve at each spectral acceleration level. The combined (mean) fragility is computed from the expected value of the probability at each spectral acceleration level. (See Figure 5.)

3.2. Procedure for Evaluating Effects of Modeling Uncertainties

In this study, we use the response surface methodology to quantify the effects of modeling uncertainties on collapse capacity. The most complete method, the full Monte Carlo procedure, is infeasible because of the computationally intensive nature of the analysis (it takes approximately 160 minutes to compute the mean collapse capacity for one set of realizations of the input random variables). The simplest method, FOSM with mean estimates approach, is unable to capture the shift in the median of the

distribution, and is insufficient to capture the effects of model uncertainties (especially where the median collapse capacity is small relative to the site seismic hazards).

Firstly, sensitivity analyses are used to probe the effects of modeling variables on the collapse capacity of the system. The results of the sensitivity analysis are used to create a response surface using regression analysis. The response surface has a second-order polynomial functional form, which is capable of representing asymmetric response to modeling (random) variables, and interactive effects between the random variables. Following creation of the response surface, Monte Carlo simulation is used to obtain a suite of sample realizations for the set of random variables under consideration. For each set of realizations, the collapse capacity of the structure is computed from the response surface. The outcome is a set of predicted collapse capacities for the structure, which represent the combined effect of modeling and record-to-record uncertainties. These results are then combined through the third approach described above.

4.   Evaluation of Effect of Modeling Uncertainties on Case Study Structures

In this study, the collapse capacities of ductile RC SMF structures of three different heights (1, 4 and 12 stories) are assessed. All structures have 20 ft. bay spacing, and have 13 ft. story heights except at the first story (15 ft.). The collapse assessment was performed using as the procedure described in Section 2, discussed in more detail in Haselton (2006). A summary of the main metrics of collapse performance is shown in Table 1. These measures include only the effects of record-to-record uncertainties associated with the variability in ground motions, and are based on results for the model with mean values for all modeling parameters.

For each of these structures, we consider uncertainty in the modeling parameters that define the lumped plasticity plastic hinges for beams and columns. These hinges are modeled using a material model developed by Ibarra and Krawinkler (2005). The backbone (Figure 1b) and hysteretic rules are defined by six parameters: flexural strength (M_y), initial stiffness, post-yield (hardening) stiffness, capping point ($\theta_{cap,pl}$), post-capping stiffness (θ_{pc}) and cyclic deterioration (λ). Each of these parameters is assumed to be lognormally distributed, and the mean and standard deviation are obtained from previous research (Haselton et al. 2007). In this study, hardening stiffness is neglected because of its very small influence on collapse capacity. For simplicity, other parameters related to element level modeling (eg. pinching and residual strength) and system level behaviour (eg. damping, mass, live and dead loading) are not considered; earlier sensitivity studies found that modeling parameters related to component strength and deformation capacity had the most significant effect on the collapse assessment (Haselton 2006). The uncertainty associated with the modeling and behaviour of reinforced concrete joints is also neglected for these structures, because capacity design provisions and transverse reinforcement requirements for joints have been shown to be sufficient to ensure that failure occurs outside the joints.

If each of the random variables discussed in the preceding paragraph were investigated individually the sensitivity analyses would quickly become extremely time intensive, requiring examination of 5 random variables for each plastic hinge location in the analytical model. To further reduce the number of variables under consideration, we make assumptions about correlations, at both the element and building level. At the element level, two meta random variables are created. The strength/stiffness meta variable assumes that strength and stiffness are perfectly correlated within the element. The ductility meta variable assumes that plastic rotation capacity, cyclic deterioration, and post-capping stiffness are perfectly correlated. These groupings are assumed, but a study of the correlations among these random variables (from the calibration results in Haselton et al. 2007) reveals that each random variable does tend to be more highly correlated with the other random variables within its group.   Further

correlations are assumed at the structural level; we assume that beam strength/stiffness is perfectly correlated over the entire structure, and likewise for column strength/stiffness, column ductility and beam ductility meta variables. These correlation assumptions leave four meta variables, each assumed to be lognormally distributed. These are all normalized random variables, and their values reflect the number of standard deviations that the realization is from the mean value.

Based on these four random variables, sensitivity analyses are conducted to quantify the effects of each modeling variable on the collapse distribution. The realizations of random variables used in the sensitivity analysis are based on central composite design, including star points (in which only one random variable is changed at a time) and factorial points (capturing interactions between the random variables) (Pinto et al. 2005). In total, 33 sensitivity analyses were conducted for each structure, and each random variable was perturbed a maximum of 1.7 standard deviations away from the mean. For each sensitivity analysis, a nonlinear model is created with modified element material properties, and the collapse analysis is run with a subset of 20 earthquake records.[5] A summary of the results of the sensitivity analysis for the 4-story SMF is shown in Figure 3. Of the four random variables, column strength/stiffness and column ductility have the largest effect. Beam strength/stiffness has an inverse effect due to the benefit having weak beams relative to the columns. It is also noteworthy that all the random variables have an asymmetric effect, ie. improvement and degradation of a capacity random variable do not have equivalent positive and negative effects on the response. This characteristic is problem for FOSM analysis, which cannot capture this nonlinearity.

The sensitivity analysis results are used to create a response surface that describes the collapse capacity $[\ln(Sa(T_1))]$ as a function of the input random variables. The response surface is the second-order polynomial that best fits the data, obtained using the regression analysis capabilities of Matlab. For the 4-story building, the fitted polynomial is partially described in Equation (1)[6]:

$$\ln(Sa(T_1)) = 0.26 - 0.08(BS) + 0.20(CS) + 0.07(BD) + 0.01(CD) - 0.05(BS^2) + ... \qquad (1)$$

where BS refers to beam strength/stiffness, BD refers to beam ductility, CS refers to column strength/stiffness and CD refers to column ductility. Since these are normalized random variables, when all meta variables are 0 all random variables are at their mean values, and the results should be consistent with the mean model; Eqn. (1) predicts exp(0.26) = 1.30g, compared to 1.30g in Table 1. A graphical representation of the response surface is provided in Figure 4. As expected, column strength/stiffness, column ductility and beam ductility all have a positive effect on the collapse capacity of the structure, while beam strength/stiffness has an inverse effect. The response surface obtained in (1) is evaluated according to statistical measures of goodness of fit. In particular, the R^2 value, which characterizes how much of the variability is captured by the regression, is 0.99. The p-value of 1.11 x 10^{-16} strongly indicates statistical significance. In addition, the variance inflation factors are computed to be << 10, indicating that collinearity is not a problem. Similar results are obtained for the 1 and 12 story SMFs.

Monte Carlo simulations are conducted using the response surfaces created for each structure. For each simulation, realizations of each of the different random variables are generated, in keeping with the lognormal assumption for the meta random variables. From these realizations, the response surface is used to obtain a prediction of the collapse capacity of the structure which the simulation represents. Ten thousand simulations were performed, and the predicted collapse capacity for each was obtained.

---

[5] This subset of earthquake records was chosen to reduce the computational times needed. The response spectra of this subset was observed to be characteristic of the response spectra of the whole set.
[6] The coefficients on higher order interaction terms are not shown here for simplicity.

The final step is to recreate the cumulative distribution of collapse, incorporating the information from the Monte Carlo simulations that includes the effects of modeling uncertainties. Each simulation predicts a value of the collapse fragility at each spectral acceleration level; by taking the expected value of these predictions the final fragility is obtained. This process is illustrated in Figure 5, for the 4-story structure. The final probability distributions for collapse capacity of the 1, 4, and 12 story structures are shown in Figure 6, and these effects are summarized in Table 2. The effect of incorporating modeling uncertainties is to decrease the prediction of the median and increase the dispersion of the collapse fragility. However, the extent of the effect depends on the structure under consideration. The base case, no consideration of modeling uncertainty, is highly unconservative. Table 2 also illustrates how choices about how to incorporate modeling uncertainty affect the prediction of the mean annual frequency of collapse. Figure 6 compares the results obtained by a simplified FOSM approach assuming $\sigma_{ln,modeling} = 0.45$, as described previously.[3] At the left tail, the FOSM-obtained CDFs (using the mean estimates approach) are relatively close to those obtained from the Monte Carlo simulation. In this study the mean annual frequencies computed using the FOSM/mean estimate approach and the full Monte Carlo/response surface approach are fairly close, but this result may be significantly different for non-ductile structures.

The Monte Carlo simulation also allows us to conduct a parametric study of the effects of correlation assumptions between the meta random variables. In the results presented thus far the four meta random variables are assumed to be uncorrelated. Two other sets of correlation assumptions are considered for the 4-story structure. In the first case BS and CS are assumed to be correlated, as are BD and CD, but there is no correlation assumed between the two groups. In the second case, BS and BD are assumed to be correlated, as are CS and CD. For the first case, full correlation leads to a 9.3% increase in the median collapse capacity, reducing the overall effect of considering modeling uncertainty. This suggests that the relative difference in beam and column strength and beam and column ductility is a larger factor in determining collapse capacity than the absolute values. In the second case, as the assumed level of correlation increases the median collapse capacity decreases (by 5.6% in the fully correlated case) and the dispersion increases. At higher levels of correlation it becomes more likely that beam behaviour is either very good (in terms of both strength and ductility) or very bad. Since poor behaviour tends to decrease the collapse capacity more than good behaviour increases it, the median is further reduced as higher levels of correlation are assumed.

5. Conclusions

The results of this study, which utilizes Monte Carlo simulation in conjunction with a fitted response surface, demonstrate that incorporation of modeling uncertainties can have a significant effect on the collapse fragility obtained. Neglecting these effects is nonconservative.

For the ductile RC moment frames considered in this study, explicit consideration of uncertainties associated with element strength/stiffness and ductility may decrease the median collapse capacity by 5 to 21%. This decrease in collapse capacity occurs because the effects of the random variables on the collapse response are asymmetric, and tend to have a larger negative than positive effect. The dispersion of the collapse fragility also increases, by between 16 and 23% for the case study structures. Variation in the importance of modeling uncertainties for the three different structures is likely due to the possible failure modes in the structure and the propensity of the modeling uncertainties to alter the most likely failure mode. In particular, the 1-story building has essentially one failure mode consisting of hinging in the column bases and hinging in the columns or beams at the 1st floor. In contrast, there are nearly ten observed failure modes for the 4-story building, and this may be associated with the larger effect of modeling uncertainties in this case. Correlations also have a significant impact on

collapse performance predictions; the correlation cases considered here show a 5 – 10% change in the median collapse capacity. Further cases of building level correlations, particularly for the 4-story building, are a topic for future research.

These results also suggest that for a ductile well-performing structure (ie. the MCE is significantly below the median collapse capacity) the simplified FOSM approach gives reasonable estimates, because the lower tails of the collapse distributions obtained in the two cases are relatively similar. The FOSM approach, however, cannot predict a shift in the median value, which is important for some structures. Moreover, for non-ductile structures which have significantly lower collapse capacities relative to the site hazard level, differences at the upper end of the collapse distribution will become more critical and the FOSM results show poor agreement with the Monte Carlo results.

These results point more generally to the potential importance of characterizing and propagating uncertainties appropriately. Because simplified approaches may have a large effect on calculated risks the accuracy of simplifying assumptions should be considered with care when the results will impact important decisions.

Acknowledgements

References

Aslani, H. (2005). "Probabilitistic earthquake loss estimation and loss disaggregation in buildings," Doctoral Dissertation, Stanford University.

Baker, J. W., and Cornell, C. A. (2007). "Uncertainty propagation in seismic loss estimation." *Structural Safety?*

Cornell, C. A., Jalayer, F., Hamburger, R., and Foutch, D. A. (2002). "Probabilistic Basis for 2000 SAC FederalEmergency Management Agency Steel Moment Frame Guidelines." *Journal of Structural Engineering*, 128 (4), 525 - 533.

Deierlein, G. G. (2004). "Overview of a Comprehensive Framework for Earthquake Performance Assessment, Performance-Based Seismic Design Concepts and Implementation, Proceedings of an International Workshop." *PEER Report 2004/05*.

Esteva, L., and Ruiz, S. E. (1989). "Seismic Failure Rates of Multistory Frames." *Journal of Structural Engineering*, 115(2), 268 - 283.

Goulet, C., Haselton, C. B., Mitrani-Reiser, J., Beck, J., Deierlein, G. G., Porter, K. A., and Stewart, J. (2007). "Evaluation of the Seismic Performance of a Code-Conforming Reinforced-Concrete Frame Building - from seismic hazard to collapse Safety and Economic Losses." *Earthquake Engineering and Structural Dynamics*, (In Press).

Haselton, C., Liel, A., Taylor Lange, S., and Deierlein, G. G. (2007). "Beam-Column Element Model Calibrated for Predicting Flexural Response Leading to Global Collapse of RC Frame Buildings (In Preparation)." Pacific Earthquake Engineering Research Center 2007/xx, University of California, Berkeley.

Haselton, C. B. (2006). "Assessing Seismic Collapse Safety of Modern Reinforced Concrete Frame Buildings," Doctoral Dissertation; Stanford University.

Haselton, C. B., Mitrani-Reiser, J., Goulet, C., Deierlein, G. G., Beck, J., Porter, K. A., Stewart, J., and Taciroglu, E. (2006). "An Assessment to Benchmark the Seismic Performance of a Code-Conforming Reinforced-Concrete Moment-Frame Building (In preparation)." Pacific Earthquake Engineering Research Center 2006/xx, University of California, Berkeley.

Helton, J. C., and Davis, F. J. (2001). "Latin Hypercube Sampling and the Propagation of Uncertainty in Analyses of Complex Systems." *SAND2001-0417*, Sandia National Laboratories.

Ibarra, L. (2003). "Global Collapse of Frame Structures under Seismic Excitations." Blume Center TR 152, Stanford University.

Ibarra, L. F., Medina, R. A., and Krawinkler, H. (2005). "Hysteretic models that incorporate strength and stiffness deterioration." *Earthquake Engineering and Structural Dynamics*, 34, 1489- 1511.

Lee, T.-H., and Mosalam, K. M. (2005). "Seismic demand sensitivity of reinforced concrete shear-wall building using FOSM method." *Earthquake Engineering and Structural Dynamics*, 34, 1719 - 1736.

Liel, A. B., Haselton, C. B., and Deierlein, G. G. "The Effectiveness of Seismic Building Code Provisions on Reducing the Collapse Risk of Reinforced Concrete Moment Frame Buildings." *4th International Conference in Earthquake Engineering*, Taipei, Taiwan.

Pinto, P., Giannini, R., and Franchin, P. (2005). *Seismic Reliability Analysis of Structures*, IUSS Press, Pavia, Italy.

Porter, K. A., Beck, J. L., and Shaikhutdinov, R. V. (2002). "Sensitivity of Building Loss Estimates to Major Uncertain Variables." *Earthquake Spectra*, 18(4), 719- 743.

Rubinstein, R. Y. (1981). *Simulation and the Monte Carlo Method*, John Wiley and Sons, New York.

Val, D., Bljuger, F., and Yankelevsky, D. (1997). "Reliability evaluation in nonlinear analysis of reinforced concrete structures." *Structural Safety*, 19(2), 203 - 217.

Vamvatsikos, D., and Cornell, C. A. (2002). "Incremental Dynamic Analysis." *Earthquake Engineering and Structural Dynamics*, 31, 491-514.

Table 1: Collapse metrics for case study ductile frame structures

Design ID	Num. of Stories	Framing System	T_1 (s)	Mean $Sa_{g.m.}$, collapse (g)	Margin compared to MCE	P[collapse| MCE]	λ_{col} x 10^{-4}	Collapse Mode (most frequently occuring)
2061	1	space	0.42	2.95	2.11	0.07	1.2	1-story collapse mechanism
1003	4	perimeter	1.12	1.3	1.71	0.09	1.7	(1) 2nd story mechanism; (2) Mechanism in stories 1 and 2
1013	12	perimeter	2.01	0.61	1.32	0.26	6.7	2; (2) Mechanism in stories 1,2 and 3

*Model with mean parameters, subset of 20 earthquake records

Table 2: Effect of modeling uncertainties on median and dispersion of collapse fragility and comparison of $\lambda_{collapse}$ with different methods of computing modeling uncertainty.

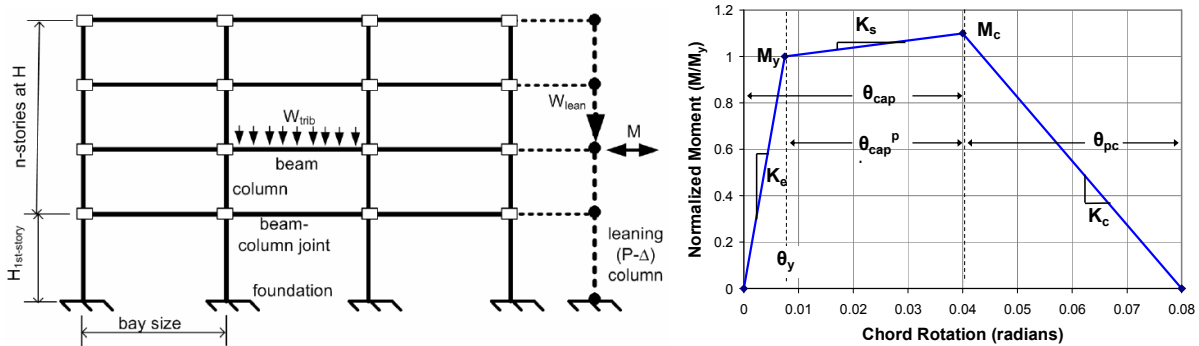| Num. of Stories	Effect of Modeling Uncertainty in this Study		$\lambda_{collapse}$ (x 10^{-4})		
	% change in median	% change in dispersion	No consideration of modeling uncertainty	FOSM[3], where $\sigma_{ln,modeling}$ = 0.45	This study
1	-5%	16%	1.2	4.1	1.6
4	-21%	23%	1.7	6.1	5.9
12	-9%	18%	6.7	17.0	12.0

Figure 1: Schematic diagram of analytical model, showing (a) generalized model configuration and (b) nonlinear material features of beam-column hinges.
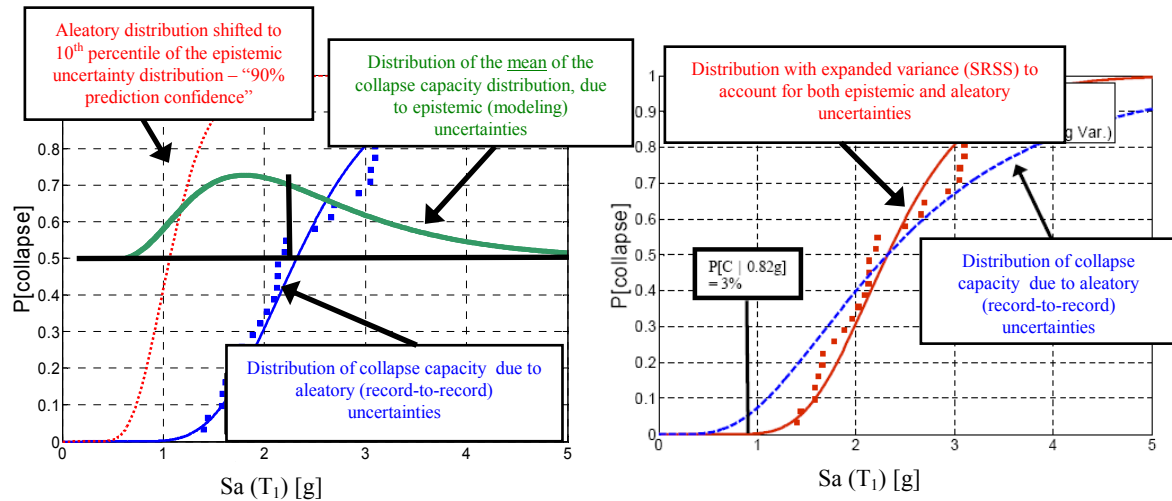
Figure 2: Collapse fragilities for a 4-RC frame structure, illustrating (a) the confidence interval approach and (b) the mean estimates approach. [After Haselton (2006)]
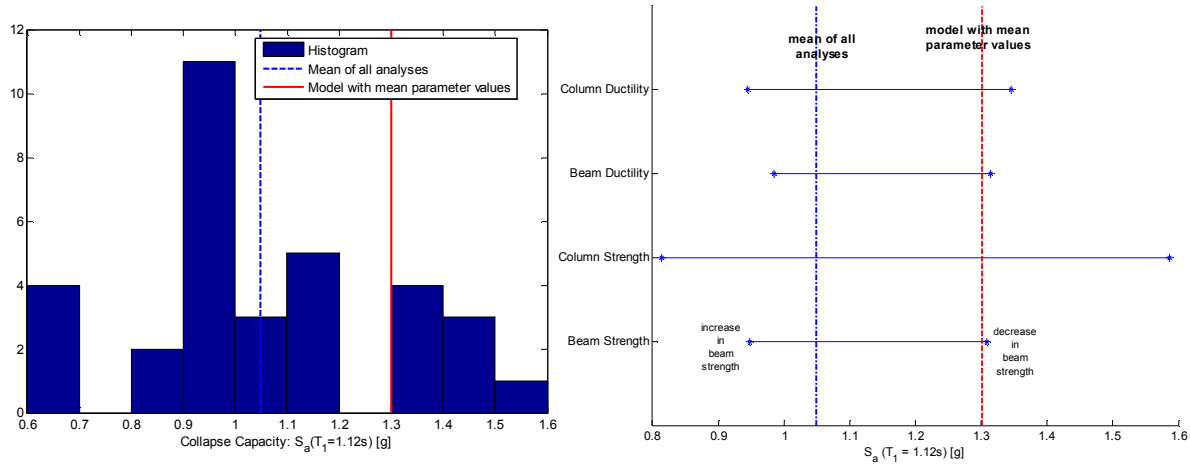

Figure 3: (a) Histogram showing the results of sensitivity analysis for the 4-story SMF. (b) Tornado diagram from sensitivity analysis results.
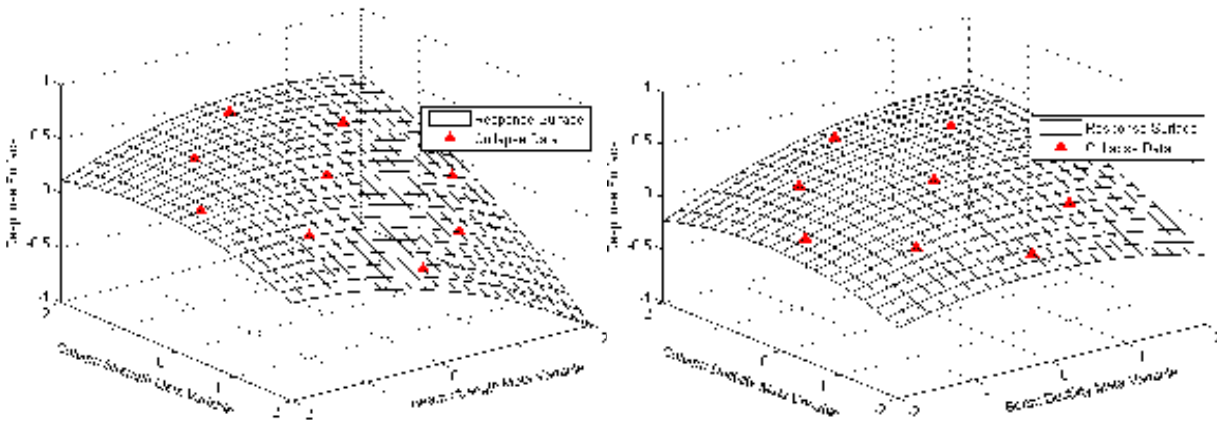

Figure 4: Graphical representation of the polynomial response surface obtained for the 4-story structure. Each of these represents a slice of a multi-dimensional surface. In (a) the effects of column strength/stiffness and beam strength/stiffness are shown, while beam ductility and column ductility are held constant (at 0). Likewise, Figure 4(b) illustrates the effects of varying beam and column ductility.
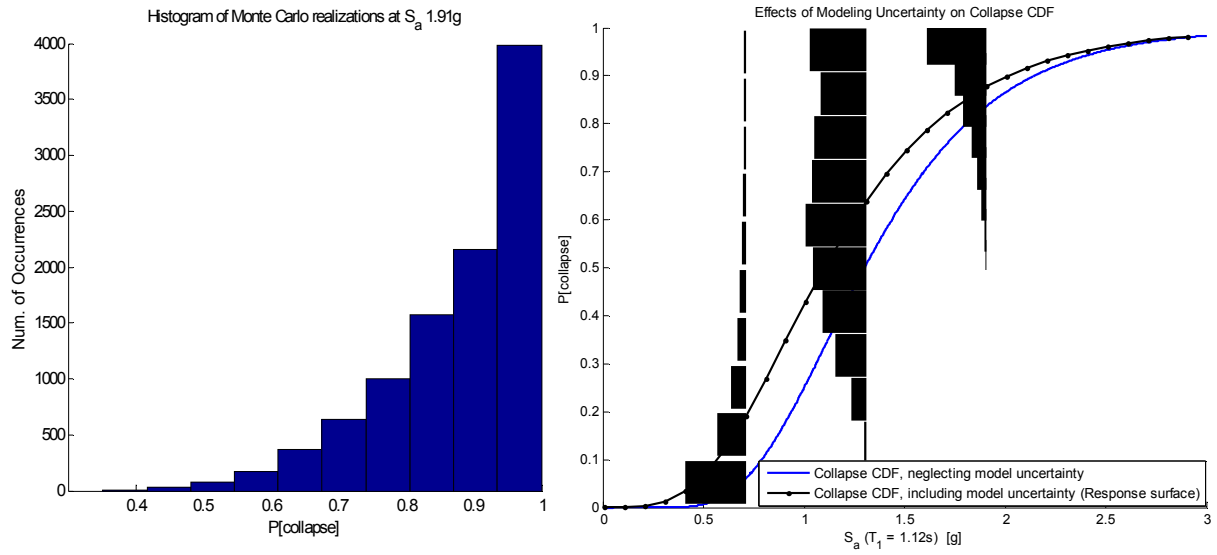
Figure 5: (a) Histogram of collapse probabilities obtained form Monte Carlo simulations at Sa = 1.91g and (b) Computed collapse CDF with histograms superimposed at selected Sa levels.
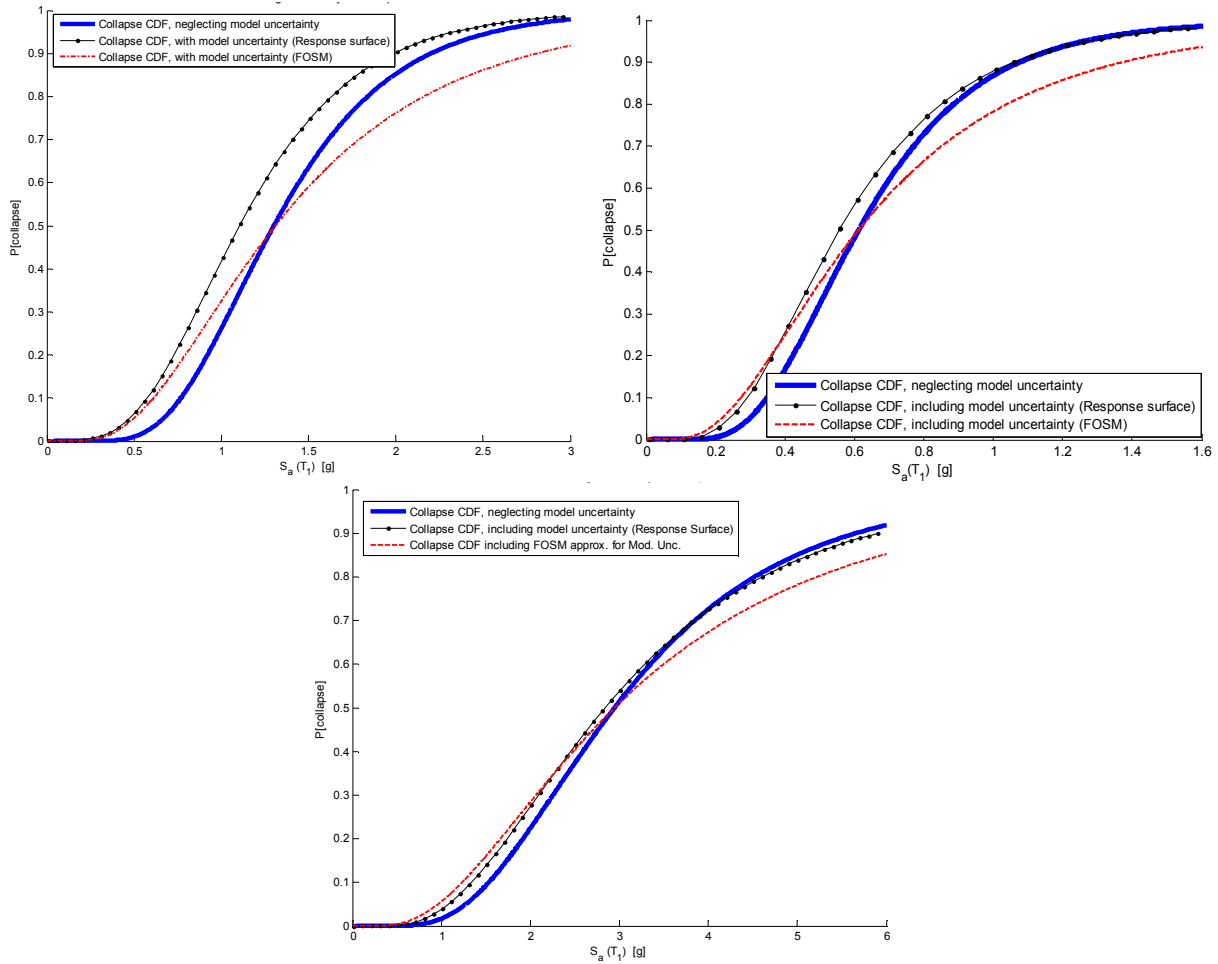

Figure 6: Cumulative CDFs obtained for (a) 4-story building, (b) 12-story building and (c) 1-story building.