Model Parameter Uncertainty Effects on the Seismic Performance of a 4-story Steel Moment-Resisting Frame

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ABSTRACT: The reliable estimation of the seismic performance of structures requires quantifying the effect of aleatory and epistemic uncertainties of the system parameters. This is efficiently achieved for a case study of a steel moment-resisting frame through several important advances. First, a state-of-the-art numerical model is formed with full parameterization of its strength properties. Empirical relationships derived from experimental data are used to model the cyclic behavior of steel sections using probabilistically distributed parameters that include intra- and inter-member correlation. Incremental dynamic analysis is employed to accurately assess the seismic performance of the model. Finally, an efficient Monte Carlo simulation algorithm is used, based on record-wise incremental Latin Hypercube Sampling to propagate the uncertainties from the numerous parameters to the actual system demand and capacity. Consequently, this modern steel frame is shown to be only mildly sensitive to parameter uncertainties when quality is tightly controlled.

1 INTRODUCTION

Several uncertainty sources come into play whenever an engineer attempts to assess the seismic performance of a structural system. The uncertainties may be organized into two main categories, these being the aleatory and the epistemic (Wen et al., 2003). The aleatory uncertainties are associated with factors inherently random, such as the earthquake loading, and hence cannot be controlled. By contrast, epistemic uncertainty is related to our incomplete knowledge and can be potentially reduced, e.g., by employing testing to determine material properties or using more sophisticated numerical models and methods of analysis.

Up until now, several recent studies (e.g. Kwon & Elnashai, 2006; Kazantzi et al., 2008a) have concluded that the earthquake "signature" is the dominant uncertainty source. However, current research has, so far, only partially addressed the issue of the uncertainties related to the parameters of the structural model in seismic performance assessment (e.g. Jeong & Elnashai, 2007; Dolsek, 2009; Liel et al., 2009; Vamvatsikos & Fragiadakis, 2010; Ibarra & Krawinkler, 2011; Kazantzi et al., 2011). For instance, Ibarra & Krawinkler (2011) have shown that the model parameter uncertainties can have a significant impact on the predicted collapse performance when considering deteriorating hysteretic models. Nevertheless, the study is limited to single-degree-of-freedom (SDOF) systems and hence the validity of the outcomes to multi-degree-of-freedom (MDOF) systems remains unresolved. By contrast, Liel et al. (2009) investigated the model uncertainty significance for a set of reinforced concrete structures that were efficiently modeled to account for cyclic deterioration. This study concluded that neglecting the modeling uncertainties reduces the dispersion in the response fragility and most commonly shifts the median in unconservative ways. Despite the revealing findings of this study, these are bounded to errors associated with the approximate nature of the response surface methodology. The latter was adopted for predicting the median collapse capacity as a function of the model random variables. On a different track, Vamvatsikos and Fragiadakis (2010) investigated the model uncertainty effects on a steel moment-resisting frame by means of Monte Carlo simulation paired with Latin Hypercube Sampling. The study concluded that the model uncertainties can have an important contribution to the overall response dispersion. Yet, generalization of the findings is limited due to the fact that the probabilistic modeling of the uncertain parameters was mostly founded on arbitrary choices rather than experimental evidences. Thus, the holistic quantification of the model parameter uncertainties and how these propagate into the analysis and performance predictions remains an open issue.

Aiming to provide such an outlook this study attempts to quantify the model parameter uncertainty for a case study of a 4-story steel moment-resisting frame (MRF), whose strength properties are fully parameterized. Empirical relationships derived from experimental data and recently proposed by Lignos & Krawinkler (2011) are used to model the cyclic behavior of steel sections via parameters that determine pre- and post-capping plastic rotation, cyclic deterioration in strength and stiffness, effective yield strength and post-yield strength ratio of steel components subjected to cyclic loading. Such variables are completely described by probabilistic distributions including intra-member and inter-member correlation information throughout the entire structure.

Incremental dynamic analysis (Vamvatsikos & Cornell, 2002) is employed to accurately assess the seismic performance of the model, for any combination of the parameters, by performing multiple nonlinear response history analyses. For these analyses a suite of ground motion records is used that is scaled to increasing intensity levels. Finally, we use an efficient Monte Carlo simulation algorithm based on record-wise incremental Latin Hypercube Sampling (LHS) to propagate the uncertainties from the model properties to the actual system demand and capacity (Vamvatsikos, 2011).

The conclusions to be reached by comparing the results obtained with and without consideration of the model parameter uncertainties are anticipated to reveal whether their effect can be safely ignored or need to be explicitly taken into account, at least for the case at hand, i.e., for regular low-rise capacity-designed buildings where good quality control is exercised during construction. Furthermore, the present study offers valuable insight into issues related to the effect of the demand-to-capacity correlation on the estimation of fragility for given limit-states.

2 LITERATURE REVIEW

Despite the fact that several studies in the past dealt with the deterioration modeling of steel frames, only limited research (e.g. Luco & Cornell, 1998; Song & Ellingwood, 1999; Kazantzi et al., 2008b; Vamvatsikos & Fragiadakis, 2010) has been focused explicitly on the model parameter uncertainty in structural component capacity as well as on how this propagates into the analysis and consequently into the seismic performance predictions. However, even in these studies, the deterioration modeling was primarily empirical and it was founded on simplified assumptions that were justified only by the need for employing the best capacity estimates given the limited available data. To this end, the dependence of the models proposed by e.g. FEMA 355D (2000), Mele (2002) and Kazantzi et al. (2008b) for estimating the steel component capacities on a single structural property (i.e. the beam depth), may be considered a step forward. Nevertheless, they have left ample space for more elaborate research towards enhanced steel structural modeling and capacity uncertainty consideration.

On account of the above, relatively recently, Lignos & Krawinkler (2011) provided detailed relationships for modeling the cyclic and in-cycle deterioration of structural steel components. The proposed multi-variable empirical equations allow the prediction of several modeling parameters on the basis of more than 300 steel wide flange beam experiments. Only beams without slabs have been considered during the selection of the experimental tests whereas experiments in which the connection type affected the plastic hinge formation were disregarded. The proposed equations are thus suitable for beams with or without reduced beam sections (RBS) where the plastic hinge forms solely within the beam length. They can be used to predict the pre-capping plastic rotation, the post-capping rotation and the cyclic deterioration, as well as the quantitative information provided for the effective yield strength and the post-yield strength ratio. These deterioration properties match the parameters of the Ibarra-Krawinkler (IK) model (Ibarra et al., 2005) as this was modified by Lignos & Krawinkler (2009) and implemented in the OpenSees open-source analysis platform (McKenna et al., 2000) to incorporate asymmetric component hysteretic behavior as well as ultimate deformation rotation.

3 CASE STUDY 4-STORY STEEL MRF

3.1 *Structural modeling*

The effect of the model parameter uncertainties on the seismic performance will be quantified by means of a case study steel building. The building consists of four stories. The first story is 4.6m (15ft) in height and the remaining three are 3.7m (12ft) high. It was designed as an office building to 2003 IBC and AISC for the Los Angeles area and it has a rectangular floor plan consisting of 3 bays at 9.1m (30ft) in the North-South direction and 4 bays at 9.1m (30ft) in the East-West direction. Our focus will be the East-West framing, in which only the two middle bays are moment-resisting. The columns of the moment-resisting bays were assumed to be fixed at their bases, whereas they are also spliced at the mid-height of the third story. The beams were designed as reduced sections (RBS) with their ‘dogbone’ geometries detailed according to FEMA 350 (2000). The MRFs are also capacity-designed, implying that the steel section sizes satisfy the AISC strong column-weak beam requirement.

The building’s seismic performance was evaluated using a 2D centreline model in OpenSees (McKenna et al., 2000) with plastic hinge formation allowed at beam as well as column ends. P-Δ effects were included using a leaning column, whereas the mathematical idealization of the frame includes shear deformation due to panel zones. The first three vibration periods of the analyzed frame were found to be 1.33, 0.40 and 0.19sec, whereas 2% Rayleigh damping was assumed at the first and third mode of vibration. Figure 1 depicts the 2-D model used for the East-West MRF along with the beam and column section sizes. Additional details regarding the frame configuration, design and idealization can be found in Lignos et al. (2011).

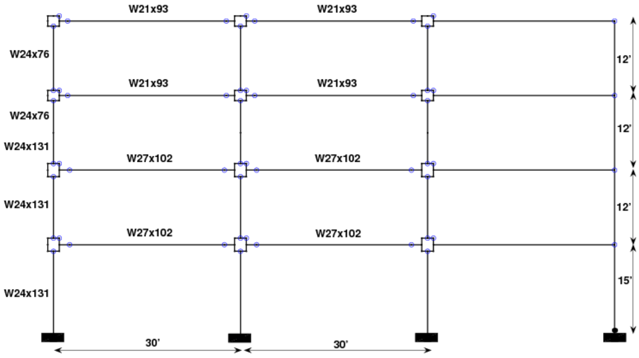


Figure 1. 2D model idealization of the analyzed East-West MRF (dimensions in feet). A leaning column has been added on the far right to simulate the effect of non-moment-resisting bays carrying mainly gravity loads.

3.2 *Probabilistic modeling*

The properties of the rotational springs at the member ends, as idealized by the modified Ibarra-Krawinkler model (see Fig. 2), completely account for material nonlinearity in the model. They are assumed to be random variables that are explicitly described by probabilistic distributions and correlated according to factors derived from experimental data. For the pre-capping plastic rotation θ_p, the post-capping plastic rotation θ_{pc} and the cumulative rotation capacity Λ in particular, the lognormal distribution was found to fit the experimental data satisfactorily based on the Kolmogorov-Smirnov test. Hence, for the present case, with respect to the RBS beam connection parameters θ_p, θ_{pc} and Λ, the 'median' (i.e. the exponential of the average of the natural logarithms of the data) and the dispersion (i.e. the standard deviation of the natural logarithms of the data) was prescribed by the empirical equations proposed by Lignos & Krawinkler (2011). Accordingly, similar equations that were proposed for other-than-RBS beams with depth $d \geq 533$mm are used for the columns as best estimates. Furthermore, for both beam and column sections the effective-to-predicted component yield strength $M_y/M_{y,p}$ as well as the capping strength-to-effective yield strength M_c/M_y ratios were assumed to be normal random variables with their properties summarized in Table 1.

Furthermore, the correlation coefficients between the random hinge properties, for each individual plastic hinge are summarized in Table 2 (Lignos & Krawinkler, 2009) and are based on the actual experimental dataset analyzed by Lignos & Krawinkler (2011).

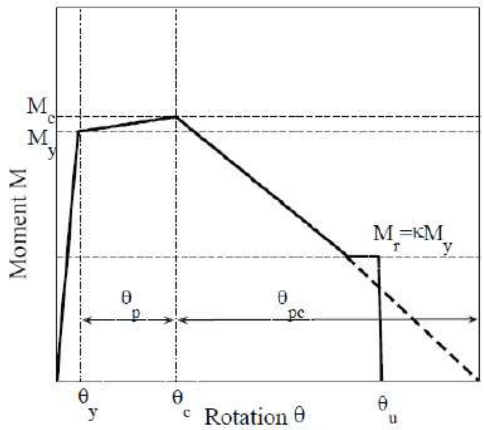


Figure 2. Monotonic moment-rotation for the modified Ibarra-Krawinkler deterioration model.

Table 1. Statistics of strength ratios.

Connection	Mean $M_y/M_{y,p}$	$\sigma_{My/My,p}$	Mean M_c/M_y	$\sigma_{Mc/My}$
RBS	1.06	0.12	1.09	0.03
Other-than-RBS	1.17	0.21	1.11	0.05

Finally, a correlation coefficient of 0.65 was assumed within variables of the same type for plastic hinges belonging to beams or columns having the same section. Such correlations have been found to characterize steel elements from the same production lot (Idota et al., 2010). Given the small size of the structure, it can be assumed that all beams and columns of the same section come from the same lot, thus relatively high inter-member correlations exist among beams at stories 1-2 and 3-4 (see Fig. 1). Similarly for the columns, US construction practice dictates that a single member crosses several stories until a splice occurrence at mid-story to change to a new section. Thus, columns are well correlated above and below the middle of the 3rd floor (Fig. 1).

Table 2. Random variables correlation coefficients for beams and columns.

Beams	θ_p	θ_{pc}	Λ	$M_y/M_{y,p}$	M_c/M_y
	1	0.54	0.65	0	0
	0.54	1	0.63	0	0
	0.65	0.63	1	0	0
	0	0	0	1	0
	0	0	0	0	1
Columns	θ_p	θ_{pc}	Λ	$M_y/M_{y,p}$	M_c/M_y
	1	0.60	0.56	0	0
	0.60	1	0.58	0	0
	0.56	0.58	1	0	0
	0	0	0	1	0
	0	0	0	0	1

To account for the uncertainties induced by the considered parameters to the structural system under examination, a recently developed Monte Carlo method paired with an efficient incremental record-wise Latin Hypercube Sampling (LHS) design is used (Vamvatsikos, 2011) to propagate the uncertainties from the numerous parameters to the actual system demand and capacity. Whereas usually a full record suite is used to analyze each model sample, this approach undertakes LHS simultaneously on the structural properties and the seismic input to achieve considerable savings. Hence, instead of maintaining the same model realization and analyzing it over the entire suite of ground motions, the latter also becomes a random variable in the sense that each model realization (or each set of structural properties) is paired to a different ground motion that is also randomly selected from a bin of records. Furthermore, the adopted LHS methodology allows us to conduct an incremental convergence process whereby the new observations are introduced by successively doubling the sample size in such a way that the existing sample can be reused.

3.3 *Incremental dynamic analysis*

Incremental Dynamic Analysis (IDA) is employed to determine the seismic response of the model structure for various combinations of the uncertain parameters. IDA (Vamvatsikos & Cornell, 2002) is a powerful analysis method that involves performing a series of nonlinear time history analyses for a suite of ground motion records, the latter scaled at increasing intensity levels. To define IDA curves of seismic intensity versus response, two scalars are needed, these being an intensity measure (IM) to represent the seismic intensity and an engineering demand parameter (EDP) to record the structural response. For the present study the 5% damped first-mode spectral acceleration $S_a(T_1, 5\%)$ is used as the IM whereas, the peak story drifts for the individual i-stories θ_i, the maximum interstory drift θ_{max} and the peak roof drift θ_{roof} are used as EDPs.

3.4 *Illustrative results*

The analyses involved a total of 200 random variables and 60 "ordinary" ground motion records (i.e. without any soft soil or directivity issues). The record-wise LHS design was applied with a starting size of 10 that was incrementally increased over 6 generations to a maximum sample of 320.

As can be inferred from Figure 3, where the relative errors in the median and dispersion in S_a terms are illustrated for two different levels of nonlinearity (i.e. 'near yield' assumed at θ_{max}=0.02 and 'near collapse' assumed at θ_{max}=0.1) and the whole set of the considered EDPs, only 4 to 5 generations at 80 to 160 samples respectively, are practically sufficient in order to achieve fairly stable response estimates. More specifically it was revealed that, after the fourth generation, the relative errors in the median and dispersion in S_a differ only slightly, irrespectively of the considered EDPs and the limit states.

Figure 4a illustrates the IDA curves, considering θ_{max} as an EDP, obtained over the 6th generation (i.e. 320 samples). It is apparent that the record-to-record variability is fairly large especially at high interstory drift demands where the building is approaching collapse. These results can be further summarized into 16,50,84% fractile IDA curves that are presented in Figure 4b. As illustrated, given for instance an $S_a(T_1, 5\%)$ of 1.0g, 16% of the samples produce approximately a $\theta_{max} \leq 3\%$, 50% of the samples a $\theta_{max} \leq 4.5\%$ and 84% of the samples a $\theta_{max} \leq 10\%$.

The most influential random variables, at least when considering θ_{max} as an EDP, were found to be the cumulative rotation capacity Λ along with pre-capping plastic rotation θ_p for the 'near yield' limit state, and the capping strength-to-effective yield strength M_c/M_y ratio for the remaining higher limit states up to the 'near collapse', defined at an interstory drift of 0.1. The extent to which these model parameters affect the seismic performance of the analyzed building can be revealed via comparison studies of the median IDAs and dispersions obtained considering the uncertainty in the structural parameters of the modeled structure (generation 6 with N=320) and a deterministic mean-parameter model. The term "mean-parameter" refers to the typical engineering approach where the model parameter uncertainty was disregarded by setting all properties to their mean (or median) value. Only record-to-record variability is considered by using the 60 ground motion records. As illustrated in Figure 5 the evaluated medians for the maximum interstory drift ratio θ_{max} are almost identical (see Fig. 5a) and the dispersions are found to be very similar (see Fig. 5b). In other words, both the bias and the variance introduced by parameter uncertainty appears to be negligible. Furthermore, the results were found to differ only marginally for either local EDPs, i.e. the individual story drifts θ_i, or global ones, i.e. the peak roof drift θ_{roof}.

Hence, given the remarkable agreement between the mean-parameter and the computationally expensive record-wise LHS model it can be said that, at least for the considered case, the model parameter uncertainty effect may be safely ignored. These observations further support findings in previous studies (e.g. Kazantzi et al., 2011) which suggested that the uncertainty associated to the acceleration "signature" is so significant that cancels out the variability associated to the structural properties, at least for well-designed buildings with good quality control during construction.

3.5 *Demand-to-capacity correlation effect on fragility estimation*

Following the evaluation of the structural demands by means of IDA, we can now embark on the estimation of the seismic fragility. The latter, is defined as a function of the conditional probability of exceeding (violating) a performance objective, given the intensity measure level (Kazantzi et al., 2008b). Such performance objectives are typically defined as a deterministic or probabilistic EDP capacity (i.e. damage threshold).

Despite the fact that numerous studies exist in the literature dealing with the seismic fragility problem, the majority adopts the hypothesis that the demand and capacity quantities are uncorrelated. The latter assumption is usually reflected at the fragility level by defining the capacity as a global limit (e.g., of roof or interstory drift) that is not directly linked to the capacities of the structural components. This is a rational choice for a structural model with deterministic properties, but not necessarily so for an uncertain structure. Then, the demand-capacity correlation becomes more or less apparent in the sense that lower member capacities lead to higher structural demands and vice versa, thus factually linking local demands and capacities.

The potential importance of such an interaction has been recognized, at least theoretically, in the SAC/FEMA framework (Cornell et al., 2002). Still, its effect on the seismic risk assessment studies and whether this can be masked under the aleatory uncertainty component has not been examined. To the authors knowledge, only relatively recently, Dolsek (2011) evaluated the seismic risk of a 4-story concrete building considering aleatory and epistemic uncertainties as well as limit states that were paired to moment and ultimate rotations at the plastic hinges. The risk for 'near collapse' was found to be more than twice as high when considering both the aleatory and the epistemic uncertainties (i.e. random population of buildings, demand-to-capacity correlation case) than the one estimated when considering only the aleatory uncertainty component (i.e. deterministic building, no demand-to-capacity correlation).

On account of the above, the importance of the demand-to-capacity correlation will be evaluated in this study on three different premises: (a) Case 1: 320 uncertain building-record pairs where the exceedance of the limit state is checked in each sample by comparing the local demands to the actual random rotational capacity of each individual hinge (full demand-to-capacity correlation), (b) Case 2: 320 uncertain building-record pairs where the exceedance of the limit state is checked in each sample by comparing the local demands to the mean rotational capacity of each individual hinge (partial demand-to-capacity correlation) and (c) Case 3: Mean model analyzed under 60 records where the exceedance of the limit state is checked by comparing the local demands to the mean rotational capacity of each individual hinge (no demand-to-capacity correlation).

For all three cases, the limit state is associated to the first exceedance of the plastic hinge rotation capacity at the end of any structural element (beam or column), the former being equal to the pre-capping plastic rotation θ_p. This condition is considered sufficient to undermine the life safety of the building' occupants and hence may be assumed to be roughly analogue to the Life Safety (LS) performance objective as per FEMA 350 (2000) definition.

Given the aforementioned, the structural fragility may be also defined as the probability that the spectral acceleration capacity random variable, S_{ac} (i.e. the minimum spectral acceleration for which the considered limit state is exceeded) being less than or equal a given spectral acceleration, S_a (Jalayer & Cornell, 2003). Furthermore, assuming a cumulative lognormal distribution to fit the spectral acceleration capacity data, we can use the 16, 50 and 84% percentiles to estimate the median as well as the dispersion of the data.

Table 3 presents the statistics of the logarithms (median and standard deviation) of the spectral acceleration capacity random variable, S_{ac}. An unconservative bias of approximately 12% in the median appears when demand-capacity-correlation is fully ignored (case 1 versus 3). Instead, the dispersions are similar across the examined range of the demand-to-capacity correlations. Note that the difference in the median predictions appears in spite of the relatively high values of inter-member correlation used and the small number of elements in each story. Larger buildings and less correlated elements would be obviously prone to far larger influence from the correlation of demand and capacity.

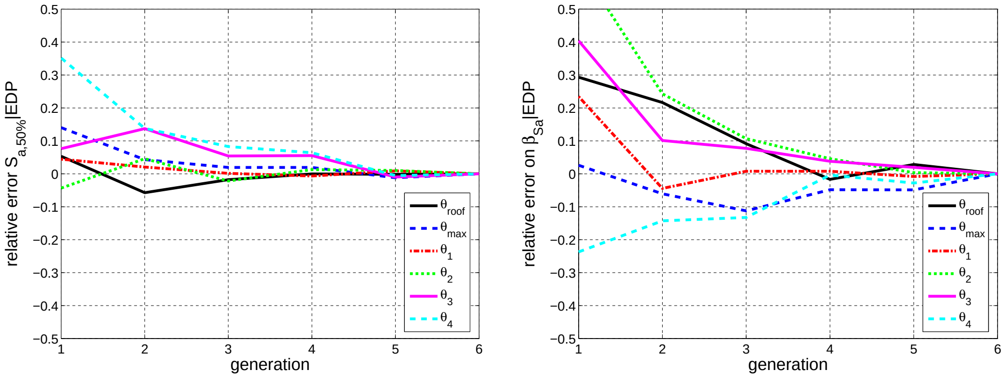
Table 3. Median and dispersion of the spectral acceleration capacity, S_{ac}, for Life Safety.

Cases	μ	β
Case 1	0.580	0.338
Case 2	0.645	0.320
Case 3	0.651	0.313

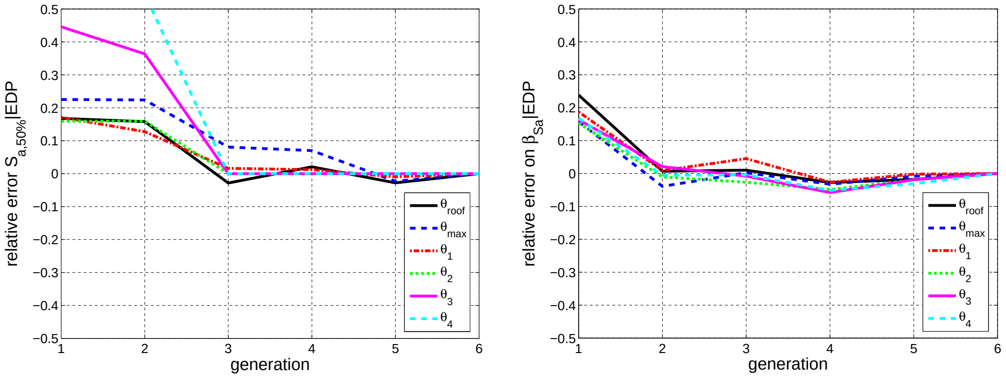
Figure 6 illustrates the fragility curves for the three examined cases that correspond to decreasing levels of demand-to-capacity correlation. For each case, the moment-fitted lognormal cumulative distribution function is plotted. By comparing the fragility curves of case 1 (full demand-to-capacity correlation) to those of cases 2 (partial demand-to-capacity correlation) and 3 (no demand-to-capacity correlation), it can be inferred that the incorporation of the demand-to-capacity correlation, results in a left shift of the fragility curve. This means that the partial and the no demand-to-capacity correlation cases (i.e. cases 2 and 3) underestimate, at least for the case at hand, the failure probabilities.

A more coherent insight into the way demand-to-capacity correlation affects the seismic fragility may be attained by inspecting the statistics of the failure locations in each one of the three considered cases. To this end, Table 4 summarizes, in percentages, the frequency with which the elements of a particular story are 'responsible' for violating the anticipated limit state (i.e. one hinge exceeding the pre-capping plastic rotation, θ_p).

As can be inferred from Table 4, the percentages associated with 'first-hinge-failures' across the building are remarkable similar for cases 1 and 2 when considering beam failures whereas, they differ considerably when it comes to column failures.

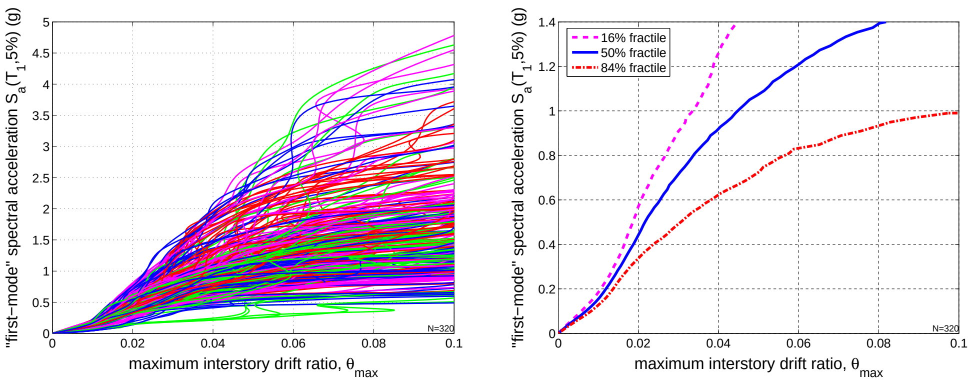


(a) $S_{a,50\%}$, ‘near yield’ (b) β_{S_a}, ‘near yield’



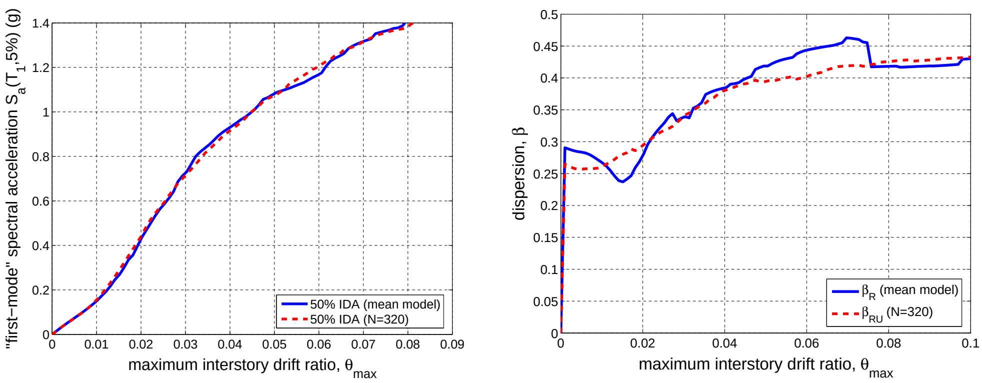
(c) $S_{a,50\%}$, ‘near collapse’ (d) β_{S_a}, ‘near collapse’

Figure 3. Relative errors in the median and the dispersion in S_a for two limit states and the considered set of EDPs.



(a) (b)

Figure 4. (a) 320 IDA curves and (b) the corresponding 16,50,84% fractile curves



(a) (b)

Figure 5. Record-wise LHS vs. mean model: (a) median IDAs and (b) S_a capacity dispersion.

In particular, approximately 13% more 'first-hinge-failures' were observed at the fourth floor columns of case 2 as opposed to case 1 and about 10% less at the first floor columns. It is believed this shift of column 'first-hinge-failures' to the top level is associated with the lack of high rotational capacity samples in case 2 that gives rise to the concentration effect of the high structural demands at the top stories for this low-rise structure. The distribution of damage locations for case 3 substantially differs compared to cases 1 and 2 with the great majority of 'first-hinge-failures' being observed in the first floor beams and the fourth floor columns.

Table 4. Percentages of 'first-hinge-failures'.

Failure location	Case 1 (%)	Case 2 (%)	Case 3 (%)
Beams 1st Floor	23.44	26.25	46.67
Beams 2nd Floor	15.63	15.00	5.00
Beams 3rd Floor	2.81	0.00	0.00
Beams 4th Floor	2.19	0.63	0.00
Columns 1st Floor	26.25	15.94	0.00
Columns 2nd Floor	0.00	0.00	0.00
Columns 3rd Floor	0.94	0.00	0.00
Columns 4th Floor	28.75	42.19	48.33

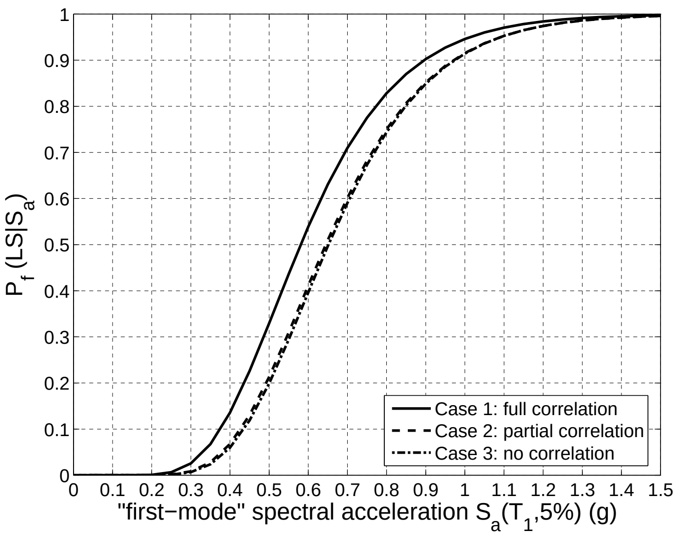


Figure 6. Fitted fragility curves for different levels of demand-to-capacity correlation.

Evidently, it is this modification of the 'first-hinge-failure' location patterns between the three cases that results in the seismic fragility shift observed in Figure 6. Furthermore, the aforementioned observations underline the potential importance of the demand-to-capacity correlation in studies related to the identification of damage patterns for an uncertain realistic building.

4 CONCLUSIONS

An accurate quantification of the model parameter uncertainty effects on the seismic performance has been presented by means of a case study steel moment resisting frame designed for urban California.

The comparison of the structural demands obtained with and without the consideration of model parameter uncertainties revealed that their effect can be safely ignored for the examined case, i.e., for regular low-rise capacity-designed buildings where good quality control is exercised during construction. For older buildings or lower quality settings, further research is needed. Nevertheless, the study also showed that, when it comes to fragility estimation and consequently seismic risk predictions as well as to studies dealing with the identification of damage patterns in a building population, caution should be exercised when ignoring the model parameter uncertainties, since the potential demand-to-capacity correlation is likely to give rise to unconservative errors.

To safely quantify the effect of the demand-to-capacity correlation, further research is needed that will account for other damage limit states, for lower correlation coefficients between the hinges of the same floor as well as for lower construction qualities. In the present study of a small structure with well-correlated members, by ignoring demand-to-capacity correlation we only observe differences in the order of 12% in the median spectral acceleration capacity. These should not be of immediate concern, yet they plant the seeds of doubt for different cases where the conditions are not as favorable.

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