THE INFLUENCE OF EPISTEMIC UNCERTAINTY ON THE SEISMIC PERFORMANCE ASSESSMENT OF BUILDINGS

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ABSTRACT :

In the paper a practical methodology for the determination of the set of structural models, which need to be defined in order to simulate the influence of epistemic uncertainty on seismic response parameters, is presented. The method is based on the Latin Hypercube Sampling, and makes it possible to take into consideration different sources of epistemic uncertainty, such as the mechanical characteristics of the materials, gravity loads and the corresponding masses, viscous damping, and other modelling uncertainties such as effective slab widths and ultimate rotations. Basically it is possible to include all types of the epistemic uncertainties, which can be simulated by means of random variables. However, it is practical to consider only a limited number of the random variables, i.e. only those which have a significant influence on the seismic response of the structure. The proposed procedure has been applied to two case study reinforced concrete structures, which were both pseudo-dynamically tested at the ELSA Laboratory, Ispra. Different sources of epistemic uncertainty, which are related to the uncertain parameters of the model and the mechanical characteristics of materials, are considered in the analysis. The influence of these uncertainties on the seismic response parameter was studied based on the simplified nonlinear seismic assessment method (N2 method) as well as by means of Incremental Dynamic Analysis. The influence of the epistemic uncertainty on the seismic demand and capacity is presented in terms of the global collapse capacity.

KEYWORDS: Performance-based earthquake engineering, uncertainty, epistemic, latin hypercube sampling, nonlinear seismic analysis, probabilistic incremental dynamic analysis, pushover analysis

1. INTRODUCTION

The performance of structures in recent catastrophic earthquakes points to the need for improved seismic design approaches capable of achieving explicit determination of seismic risk. Therefore it is important to consider that mathematical modelling of the seismic response is a subject of many modelling and physical uncertainties, which can, in addition to the aleatory uncertainty, significantly influence on the seismic response of building structures. Different methods are available for studying the influence of epistemic uncertainties on the seismic response parameters. For example, the sensitivity of single input variable on the seismic response parameter is the simplest approach for estimating the importance of the epistemic uncertainty (Porter et al. 2002). Results of the sensitivity analysis can be used for the first-order second-moment (FOSM) reliability analysis in order to estimate the effects of several modelling uncertainties on the structural response parameters. The FOSM reliability analysis was used by Haselton (2006) for studying the effects of modelling uncertainties on the collapse capacity of reinforced concrete frames designed for a high seismic region in California or, for example, by Lee and Mosalam (2005). They studied sensitivity of seismic demand to possible future earthquakes for a reinforced concrete shear-wall building using FOSM method in combination with Monte Carlo simulation. On the other hand Baker (2008) used FOSM method in combination with numerical integration for propagation of uncertainties in probabilistic seismic loss estimation. Unfortunately, the FOSM method may become inaccurate for highly nonlinear problems (Liel et al. 2008). The alternative in such cases is Monte Carlo simulation, which is computationally extremely demanding, but has an advantage of direct incorporation of modelling uncertainty into the problem.

The incremental dynamic analysis (IDA) proposed by Vamvatsikos and Cornell (2002) tends to be the most popular method for prediction of the seismic response parameters, which are determined by considering only the aleatory uncertainties (record-to-record variability). Recently this method was extended by introducing the set of structural models (Dolšek 2008a) in addition to the set of ground motion records, which is employed in the IDA analysis in order to capture record-to-record variability. The set of structural models reflects the epistemic (modelling) uncertainties and are determined by utilizing the Latin Hypercube Sampling method, which is a special type of Monte Carlo simulation.

2. SUMMARY OF THE PROBABILISTIC INCREMENTAL DYNAMIC ANALYSIS

The main steps of the probabilistic IDA analysis are presented in Figure 1 (Dolšek 2008a). Extension of the IDA analysis is straightforward since the only difference between the probabilistic IDA analysis and the IDA analysis introduced by Vamvatsikos and Cornell (2002) is the determination of the set of structural models. Once the set of structural models is determined the single-record IDA curves can be calculated for each ground motion record and for each structural model defined by the set of ground motion records and by the set of structural models, respectively. Note that exactly the same algorithms as suggested by Vamvatsikos and Cornell [1,2] can be used to determine the single-record IDA curves. The probabilistic IDA analysis is therefore more time-consuming since the IDA curves are calculated not only for the different ground motion records but also for predefined set of structural models. However, it is still less computationally demanding than a corresponding Monte Carlo simulation.

The first step in the process of the determination of the set of structural models is the identification of the sources of epistemic uncertainty. Basically, it is possible to include all types of the epistemic uncertainties, which can be described by means of random variables. If the number of random variables considered in the process of determination the set of structural model is low also the size of the set of structural models, usually referred as the number of simulations N_{Sim}, can be low. In general, two steps, the sampling of each random variable and the minimization of the difference between the prescribed and the generated correlation matrix, have to be performed in order to define the required sample of random variables. Detailed description of the determination of the set of structural model can be found elsewhere (Dolšek 2008a).

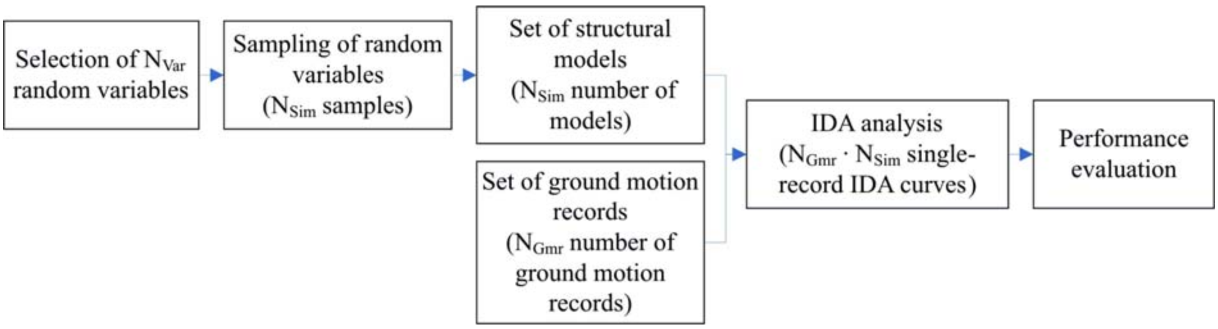

Figure 1. The main steps of the probabilistic IDA analysis.

3. CASE STUDY STRUCTURES AND GROUND MOTION RECORDS

The collapse capacity was determined for two case study structures, which are presented in Figure 4. The first structure is a 4-storey ductile reinforced concrete (RC) building (Figure 4a), which has been designed according to early versions of Eurocodes 2 and 8 (Fardis (ed.) 1996). For this structure different pseudo-dynamic tests has been performed at the European Laboratory for Structural Assessment (ELSA, Ispra) (Negro et al. 1996). The design base shear versus the weight of the structure corresponded to about 16% (Fardis (ed.) 1996). The second structure is a 4-storey plane RC frame (Figure 4b), which had been designed to reproduce the design practice in southern European countries about forty to fifty years ago (Carvalho and Coelho (Eds.) 2001). The design base shear versus the weight of the non-ductile frame is only one half of that for the ductile frame. Also this structure was pseudo-dynamically tested at full scale at the ELSA Laboratory. The results of the experiments can be found elsewhere (Carvalho and Coelho (Eds.) 2001).

The model of the case study structures consists of one-component lumped plasticity elements, which were used for modeling of the beams and columns. The zero moment point was assumed to be at the mid-span of the columns and beams. The moment-rotation relationships of plastic hinges were determined according to the procedure described by (Fajfar et al. 2006). A trilinear moment–rotation relationship for plastic hinges in columns and beams, with an initial elastic part corresponding to a cracked cross section, a second part representing yielding, and a strength degrading part after the NC limit state, was assumed for both case study structures. The ultimate rotation Θ_u in the columns at the near collapse (NC) limit state, which corresponds to a 20% reduction in the maximum moment, was estimated by means of the CAE method (Peruš et al. 2006) and according to the EC8-3 (CEN 2005) formulas for the columns and beams, respectively. All analyses were performed by the OpenSees (McKenna et al. 2000) in combination with the OS Modeler (Dolšek 2008b), which enables creating the input files for OpenSees and post-processing of the analysis results. Structural models, which were developed by assuming the best estimates for input parameters, were validated with the experimental results (Dolšek 2008c).

Fourteen ground motion records from the European Strong Motion Database and the EC8 (CEN 2004) elastic spectrum for soil type B were assumed for seismic loading in the case of the dynamic analysis and in the case of the simplified seismic performance assessment of structures by using the N2 method (Fajfar 2000), respectively. The acceleration spectra for each ground motion record and the mean spectrum are presented in Figure 8. All records are recorded on the stiff soil and the peak ground acceleration exceeds 0.1g. Since there few strong records are available in the database (Ambraseys et al. 2000), these were the only criteria for selection of the set of ground motion records.

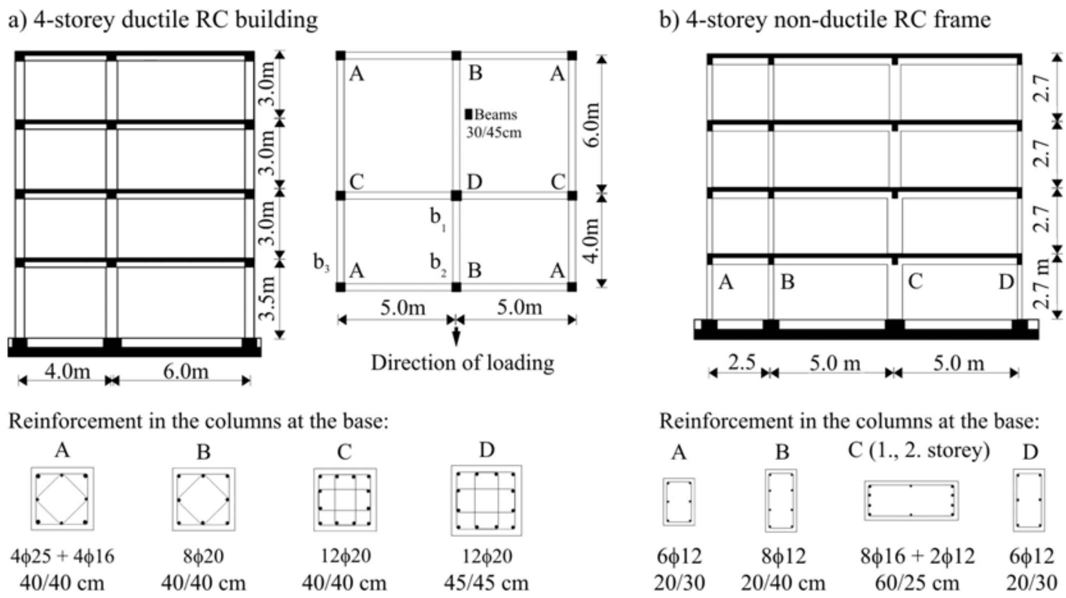

Figure 3. The plan and the reinforcement in the columns at the base for a) 4-storey ductile RC building and b) 4-storey non-ductile RC frame.

4. ASSESSMENT OF THE COLLAPSE CAPACITY

The influence of epistemic uncertainty on seismic response parameter is presented by estimating the collapse capacity of the example structures. The collapse capacity, expressed by the mean peak ground acceleration, was determined using the N2 method and IDA analysis. In the case of the IDA analysis the collapse is defined at the global dynamic instability. In this range an IDA curve becomes horizontal. Since the N2 method, as implemented in Eurocode 8, can not be used for assessing the global dynamic instability of the structure, the collapse was conservatively assumed as minimum of the top displacements, which correspond to the 20% reduction of the maximum strength and at which the rotation in the first plastic hinge of column exceeds ultimate rotation as defined in the Section 3. For that reason the collapse capacity estimated with the N2 method is conservative and therefore not directly comparable to that determined with the IDA analysis. However, for the sake of simplicity the term collapse capacity is used for both methods.

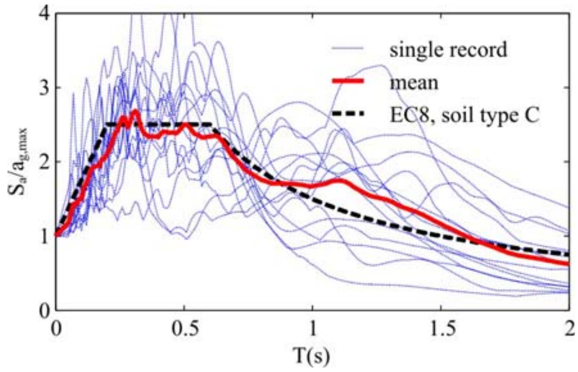

Figure 4. The single-record spectra, the mean spectrum for the selected set of ground motion records, and the EC8 spectrum for soil type B.

All random variables, which were used for determination of the set of structural models, are presented in Table 1. Mass was the only source of uncertainty, which was modeled with more than one random variable, since it was assumed that it can vary from storey to storey. All the input random variables considered for the determination of the set of structural models were assumed to be uncorrelated. The coefficients of variation presented in Table 1 were taken from literature. The highest value (0.64) was adopted for the prediction of the ultimate rotations in the beams, whereas the smaller value was used for the columns (0.4), since more reliable model was employed for determination of the ultimate rotations in columns (Peruš et al. 2006). Quite high a value for the coefficient of variation was also adopted for the initial stiffness of the beams and columns, and for the damping, which was modeled to be a random variable only in the case of IDA analysis, and it was assumed being proportional to the tangent stiffness. Note that in the case of N2 analysis seismic loading was defined with EC8 elastic spectrum for 5% damping. For the sake of brevity, discussion regarding the dispersion of the other input random variables has been omitted, since the coefficient of variation is substantially smaller. The random variables were sampled using the latin hypercube sampling method (Vorechovsky and Novak, 2003). The selected sample size is 20, since in this case the sample is large enough for simulating the prescribed correlation between random variables, which was in our case assumed to be 0. Note that some other parameters of the structural models, such as elastic modulus, gravity load, axial forces in columns, used for determination of moment-rotation relationships of plastic hinges, were linked to the appropriate random variables.

The pushover analysis was performed for both structures and for all (twenty) structural models (probabilistic model) in order to assess the global collapse capacity with the N2 method. The distribution of the lateral loads for the pushover analysis was determined as the product of the storey masses and the first mode shape. Twenty pushover curves were obtained for each structure, as shown in Figure 5. The pushover curve for deterministic models, for which the mean/median values was assumed for the random variables, and the top displacement capacity, which was defined before, are also presented. Large variability in the top displacement capacity can be observed for the ductile and non-ductile structure. The pushover curves of the deterministic models do not capture the overall response of the probabilistic models. In the case of the non-ductile structure the top displacement capacity determined with the deterministic model is underestimated in comparison to the mean top displacement capacity based on the probabilistic. The opposite was observed in the case of the ductile structure. Consequently, the collapse capacity ($a_{g,C}$) estimated with the N2 method (Fajfar, 2000), differs, if computed based on the deterministic model or probabilistic model. In our case, the $a_{g,C}$ for deterministic model of the non-ductile and ductile structure is, respectively, 0.28 g and 1.78 g. For the non-ductile structure this result is less than the mean $a_{g,C}$=0.37 g resulting from the probabilistic model. The opposite was observed for ductile structure for which the mean $a_{g,C}$=1.41 g. The dispersion β_{agC} for the collapse capacity ($a_{g,C}$) is equal to 0.19 and 0.28, respectively, for non-ductile and ductile structure. In this case the dispersion is only the consequence of epistemic uncertainty.

Table 1. The statistical characteristics of the input random variables used for determination of the set of structural models for a) ductile RC building and b) non-ductile RC frame. Note that damping was modeled as random variable only in the case of IDA analysis.

Name of variable	Unit	a) ductile building	b) non-ductile frame	COV	Distribution
mass 4th storey	t	83	40	0.1	normal
mass 3rd storey	t	86	46	0.1	normal
mass 2nd storey	t	86	46	0.1	normal
mass 1st storey	t	87	46	0.1	normal
concrete strength	MPa	32 ÷56	16	0.2	normal
steel strength	MPa	570	343.6	0.05	lognormal
effective slab width	cm	70 ÷150	75 or 125	0.2	normal
damping	%	2		0.4	normal
initial stiffness of the columns	$\Theta_{y,c}$	1·computed		0.36	normal
initial stiffness of the beams	$\Theta_{y,b}$	1·computed		0.36	normal
ultimate rotation of the columns	$\Theta_{u,c}$	1·computed		0.4	normal
ultimate rotation of the beams	$\Theta_{u,b}$	1·computed		0.6	normal

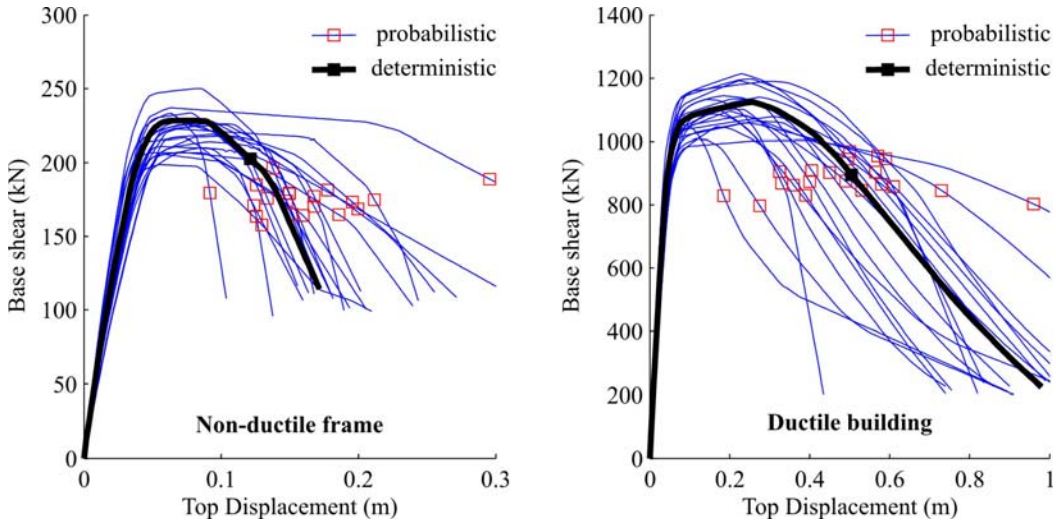

Figure 5. The pushover curves for the set of structural models (probabilistic model) compared to the pushover curve for the deterministic model. Pushover curves are presented for the non-ductile and ductile frame. The top displacement capacity is also indicated.

The influence of epistemic uncertainty on the seismic response parameters were determined also by incremental dynamic analysis. In this case the peak ground acceleration and the maximum drift was defined for the intensity measure and engineering demand parameter, respectively. A hunt and fill tracing algorithm was used to calculate the IDA curves. The collapse capacity $a_{g,C}$, was determined with a tolerance of 0.005 g. The IDA analysis was performed for probabilistic model and also for deterministic model. In the later case the IDA curves were computed for one (deterministic) model and for a set of ground motion records (Section 3), whereas in the case of probabilistic model, the IDA curves were calculated for a set of ground motion records and for each structural model from a set of structural models. The results of IDA analysis for deterministic model, for probabilistic model (probabilistic IDA), and for both structures, are presented in Figures 6 and 7, respectively. Although there is no significant difference between the two types of IDA curves, substantially higher scatter can be observed in the case of the IDA points and especially for the capacity points, if the results of the probabilistic IDA analysis (Figure 7) are compared with the results of the IDA analysis (Figure 6). However, the probabilistic summarized IDA curves practically do not deviate from the summarized IDA curves, which are determined by employing the deterministic model (Figure 8). This observation is valid mostly for the 16 and 84% fractile IDA curves the non-ductile structure and for 50% fracile IDA curve of the ductile structure. This is an interesting result, which leads to the conclusion, at least for the presented example, that the epistemic uncertainties do not significantly influence the summarized seismic response parameters, at least in the range, which is not near the collapse capacity. However, if epistemic uncertainties are considered in the analysis, median collapse capacity is reduced in the case of the non-ductile structure, and increased for the 16% fractile IDA curve of the ductile structure. The median collapse capacity estimated with the IDA and probabilistic IDA analysis is higher if compared to that obtained with the N2 method. The difference is more obvious in the case of non-ductile structure. The main source of this difference is the conservative definition of the displacement capacity if collapse capacity is determined with the N2 method (Section 3).

The important results of the IDA analysis are also the dispersion measures, which are needed for probabilistic performance evaluation. The results of IDA analysis (deterministic model) can be used only for determination of dispersion measure for randomness (R), whereas the probabilistic IDA analysis (probabilistic model) makes it possible to determine the dispersion measures which reflect randomness and epistemic uncertainty (RU), and also the dispersion measures which are caused only by the uncertainties (U). In the latter case, the dispersion measures are calculated on the basis of IDA curves for the different structural models given the ground motion record. They therefore differ from record to record. Note that terminology for the dispersion measures was adopted according to Cornell et al. (2000). For our example dispersion measures were calculated only for the peak ground acceleration, which corresponds to the collapse points (Figures 6 and 7). In all cases the dispersion was defined as the standard deviation of the natural logarithm, which was calculated as the average value of the β_{16}=log(y_{50}/y_{16}) and β_{84}=log(y_{84}/y_{50}), where y_{16}, y_{50}, y_{84} represent the counted 16%, 50% and 84% fractile in terms of the peak ground acceleration, corresponding to the collapse points. No significant difference was observed for the dispersion for randomness in collapse capacity if compared to that obtained for the non-ductile (β_{agCR}=0.68) and ductile structure (β_{agCR}=0.60). On the other hand the dispersion for the epistemic uncertainty of the non-ductile frame (β_{agCR}=0.52), expressed as the mean dispersion for randomness for a given ground motion records, significantly exceeds that of the ductile building (β_{agCR}=0.31). This difference is also expressed in the dispersion for randomness and uncertainty (β_{agCRU}), which is 0.79 and 0.66, respectively, for the non-ductile and ductile structure.

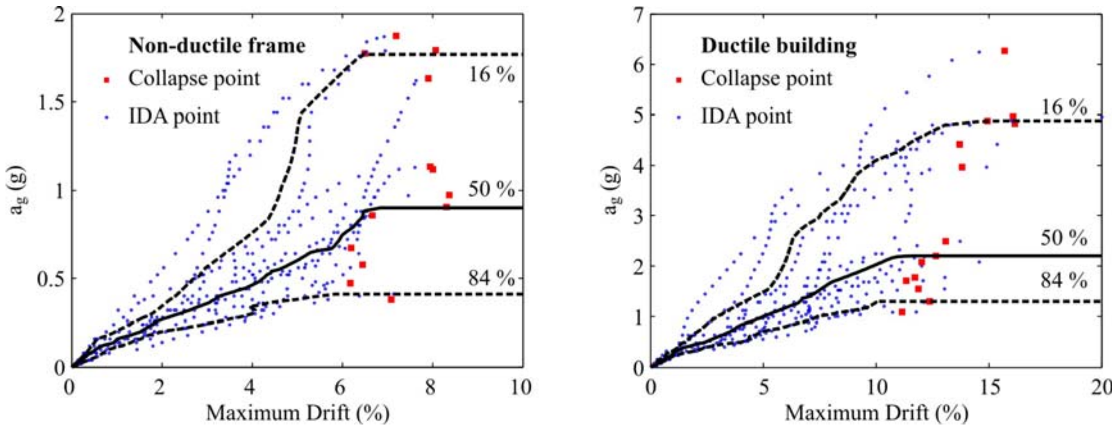

Figure 6. The summarized IDA curves, collapse and IDA points for the non-ductile and ductile structure. The IDA curves are calculated for the deterministic model for all ground motion records in a set.

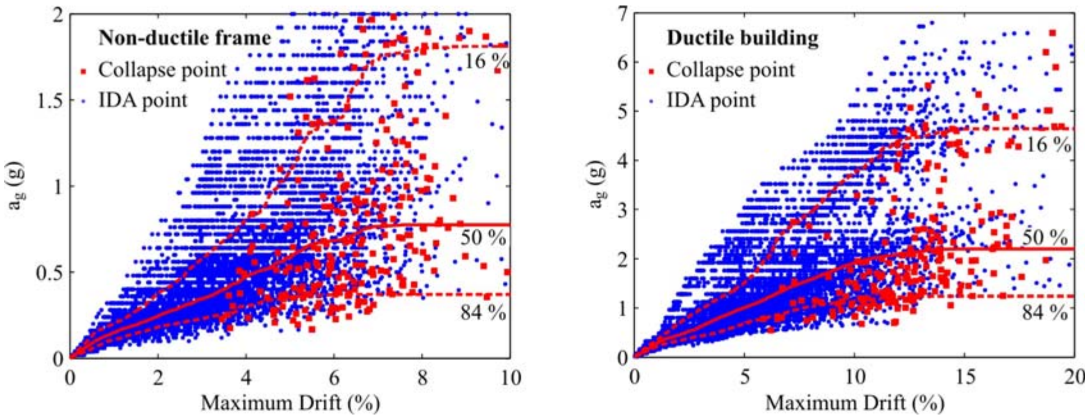

Figure 7. The summarized probabilistic IDA curves, the IDA points and the collapse points for the non-ductile and ductile structure.

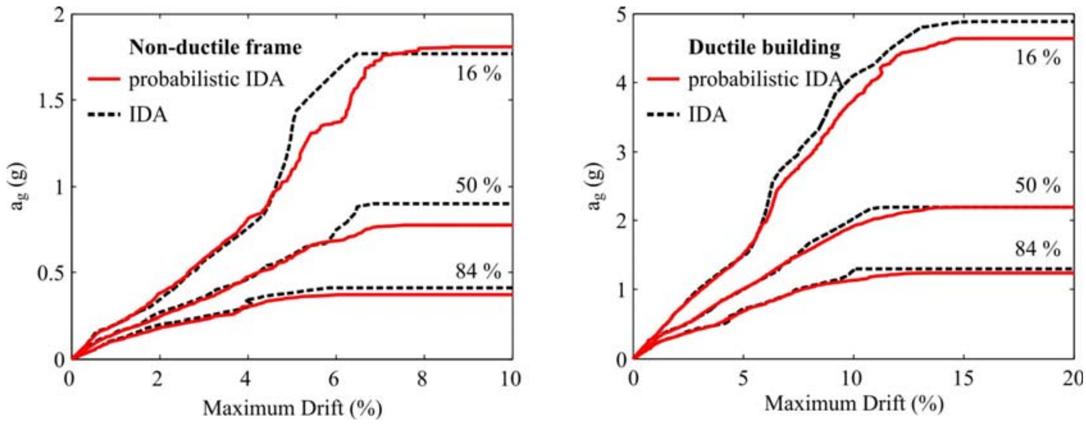

Figure 8. The summarized IDA curves based on the IDA and probabilistic IDA analysis for the non-ductile and ductile structure.

5. CONCLUSIONS

The influence of epistemic uncertainty on seismic response parameters was studied by using probabilistic IDA analysis, which combines IDA analysis and the Latin Hypercube Sampling (LHS) technique. The LHS technique was used to define the set of structural models for a non-ductile and ductile structure. These sets of structural models reflect the epistemic uncertainty, and were also used to determine the collapse capacity using the N2 method. Based on the results of the analysis, it was shown that epistemic uncertainty does not significantly influence the summarized seismic response parameter if it is determined by probabilistic IDA analysis. However, when the epistemic uncertainties were considered in the analysis, the median collapse capacity was reduced in the case of the non-ductile frame. A difference in the collapse capacity was observed also in the case of the ductile structure. In this case the 16% fractile collapse capacity was slightly increased if epistemic uncertainties were considered in the analysis. The difference in collapse capacity, if determined based on the deterministic and probabilistic model using the N2 method, was larger then that resulting from the IDA analysis. Additionally, different trends were observed if comparing the difference in collapse capacity determined by the two methods. In the case of the N2 method, the collapse capacity obtained for the probabilistic model was larger to that obtained for the deterministic model of the non-ductile structure, and smaller in the case of ductile structure. This is the opposite conclusion to that obtained in the conclusion resulting from the probabilistic IDA analysis. The reason for different conclusions can be found in fact that the collapse mechanism determined by means of pushover analysis of the deterministic model is not a good representative of the “median” collapse mechanism of the probabilistic model. Additionally, the collapse mechanism obtained from IDA analysis can significantly differ from those obtained when using pushover analysis.

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