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FAST SEISMIC PERFORMANCE ASSESSMENT OF RC FRAME STRUCTURES WITH CONSIDERATION OF ALEATORY AND EPISTEMIC UNCERTAINTY BY UTILIZING PBEE TOOLBOX AND WEB APPLICATION FOR PREDICTION OF IDA CURVES

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Abstract. *A simplified methodology for seismic performance assessment of structures with consideration of both aleatory and epistemic uncertainty is briefly presented. The methodology involves pushover analysis for a set of structural models and nonlinear dynamic analysis of corresponding equivalent SDOF models. However, a methodology by itself without any support of sophisticated computational software is not intended to be used for practical applications. Thus, a very efficient software tool, a PBEE toolbox [1] in conjunction with OpenSees, was used in our study in order to perform pushover analyses of required computational simulations, while nonlinear dynamic analysis are approximately computed by using a web application for prediction of IDA curves [2], which was recently developed within ICE4RISK project. Presented methodology and software tools are demonstrated by means of an example of a three-storey reinforced concrete frame building. The results of the presented example have indicated that incorporation of the epistemic uncertainty, in addition to aleatory uncertainty, slightly increases dispersion and can substantially decrease the limit-state intensities. This effect increases with the severity of the limit state. It was also proved that sophisticated software tools are important ingredient of performance-based earthquake engineering and can significantly facilitate transferring knowledge into practice.*

1 INTRODUCTION

Past investigations have shown, e.g. [1, 2], that accuracy of seismic performance assessment of structures can be reasonably improved if different sources of uncertainty are systematically incorporated within evaluation process. Thus, it is important in seismic performance evaluation to consider the effects of aleatory uncertainties, which are usually associated with random nature of earthquakes, and also the effects of epistemic uncertainties arising from incomplete knowledge of physical and modelling characteristics of structure.

Recently, several studies have been performed with a focus on comparisons between the different methods for incorporating the epistemic and aleatory uncertainties in seismic performance assessment, such as that performed by Liel at al. [3], or with the aim of defining simplified nonlinear methods for the evaluation of structural seismic performance, e.g. [4]. Nevertheless, a comprehensive seismic performance assessment with a systematic consideration of aleatory and epistemic uncertainties is from the computational point of view still very time-consuming due to numerous number of the required computational simulations, especially, if seismic response parameters are assessed by means of nonlinear dynamic analysis, such as that in [5, 6, 7]. It is therefore important to develop and use simplified procedures, which are computationally not excessively demanding, and are capable of sufficiently accurate prediction of the seismic response parameters with consideration of both types of the uncertainties.

For these reasons, this paper provides an overview of the procedure for fast seismic performance assessment of structures, which involves approximate IDA by using the web application [8], and allows incorporating the effect of both the aleatory and epistemic uncertainties. The use of the proposed methodology is demonstrated by the example of three-storey reinforced concrete (RC) frame building, which had been designed according to the Eurocode 8 requirements.

2 METHODOLOGY

The methodology for fast seismic performance assessment of buildings, which is explained in this section, involves the pushover analysis of the structure and the dynamic analysis of the equivalent single-degree-of-freedom (SDOF) model. The results of dynamic analysis are obtained by using the web application for prediction of IDA curves [8], which involves response database of SDOF model and n-dimensional linear interpolation. The effects of epistemic uncertainties are considered, by applying the pushover analysis and web application to the set of structural models, determined by utilizing the Latin Hypercube Sampling (LHS) technique [9].

2.1 Determination of set of structural models

In this study a variant of the LHS technique, which was recently proposed by Vorechovsky and Novak [9], was used. In general, two steps are needed to determine the sample of random variables, which are directly applied to the structural model. Firstly, each random variable X_i is sampled by using N_{Sim} values. The j-th sample value of the i-th random variable X_i can be obtained as follows:

$$x_{j,i} = F_i^{-1}\left(p_{j,i}\right) = F_i^{-1}\left(\frac{\pi_i\left(j\right)-0.5}{N_{Sim}}\right), \quad i=1,\ldots,N_{Var}, \quad j=1,\ldots,N_{Sim}, \tag{1}$$

where $\pi_i(j)$ is a random permutation of $1,\ldots, N_{Sim}$, $p_{i,j}$ is the probability that the random variable X_i is less than or equal to $x_{j,i}$ and $F_i^{-1}(p_{j,i})$ is the inverse of the cumulative distribution function (CDF) of the random variable X_i, evaluated at the probability $p_{j,i}$. The undesired correlation between the different random variables, which is introduced in the described sampling process, can be minimized by the stochastic optimization method called Simulated Annealing (see [9]). For this purpose the norm E, which is a measure for the difference between the generated and the prescribed correlation matrix, is defined by the expression:

$$E = \frac{2}{N_{Var}\left(N_{Var}-1\right)}\sqrt{\sum_{i=1}^{N_{Var}-1}\sum_{j=i+1}^{N_{Var}}\left(S_{i,j}-K_{i,j}\right)^2}, \tag{2}$$

where $S_{i,j}$ and $K_{i,j}$ are, respectively, the generated and prescribed correlation coefficients between the random variables X_i and X_j. The norm E takes into account the deviations between all the correlation coefficients, and is normalized with respect to the total number of all the correlation coefficients. It therefore represents a good measure when examples with a different number of random variables are compared. The norm E is then minimized by the stochastic optimization method known as Simulated Annealing. The result of this optimization is the optimized sample matrix **X** with N_{Sim} rows and N_{Var} columns, for which the correlation matrix is close to the target correlation matrix. More details about the LHS technique and its application can be found elsewhere (e.g. in [5, 9]).

The sample of random values is then used to generate a set of N_{Sim} structural models, which reflect the epistemic uncertainties, so that the set represents the probabilistic structural model. In a previous study [5], it was shown that if N_{Sim} slightly exceeds N_{Var}, then the optimized correlation matrix is close to the prescribed correlation matrix, and, usually, the seismic response parameters are in this case predicted with a sufficient accuracy.

2.2 Summary of the web application for prediction of IDA curves

The web application for prediction of approximate IDA curves of reinforced concrete structures, which was recently developed [8], involves response database of the equivalent single-degree-of-freedom (SDOF) model, which was computed for 30 ground motion records used by Vamvatsikos and Cornell [10].

The SDOF model was defined to be representative for reinforced concrete structure. In this case the force-displacement relationship is described by the four parameters (r_v, r_h, μ_u, α) of the pushover curve. The other two parameters of the SDOF model are the period T_1 and the ratio of the critical damping ξ. The four parameters of the force-displacement relationship are dimensionless quantities and are defined as

$$r_v = \frac{F_1}{F_2}, \quad r_h = \frac{u_1}{u_2}, \quad \mu_u = \frac{u_3}{u_2}, \quad \alpha = -\frac{k_{pc}}{k_i}, \tag{3}$$

where points (u_1, F_1) and (u_2, F_2) represent first and the second characteristic point of the idealized force-displacement relationship (Figure 1) and, respectively, roughly represent the cracking of concrete and, in the case of regular structures, yielding of reinforcements at the base of columns. The displacement u_3 is related with the displacement where the strength of the structure starts degrading, while k_{pc} and k_i are, respectively, the post-capping and initial stiffness of the idealized force-displacement relationship. With a suitable variation of the four parameters the idealized curve can be fitted to almost any pushover curve typical for reinforced concrete structures.

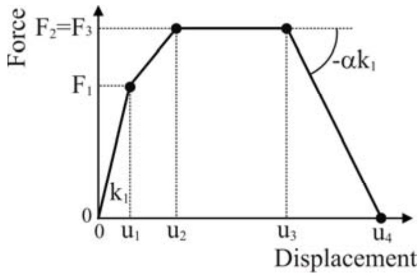

Figure 1: The idealized force-displacement relationship.

The web application was developed on the basis of the classic three-tier client-server architecture [8], which enforces a general separation of three parts: client tier (also named presentation layer or, more specifically, user interface), middle tier (business logic) and data storage tier.

The advantage of the web application for prediction of approximate IDA curves in comparison to other simplified approaches, which are based on limited parametric studies, is that the response database can be expanded by adding results of seismic response of SDOF models for additional ground motion records. Further, web application enables quadrilateral idealization of pushover curve and prediction of global dynamic instability as well as the dispersion measures, which are needed for estimation of seismic risk. These parameters are rarely estimated by simplified methods.

3 EXAMPLE

The use of the presented methodology for seismic performance assessment of structures is demonstrated by means of an example of a three-storey RC frame building, designed for the earthquake resistance according to the Eurocode 8 [11]. The seismic performance of the building is evaluated considering both the aleatory uncertainties due to the random nature of the seismic load, and the epistemic uncertainties, which relate to several sources of the uncertainty in physical characteristics of the structure and its modelling parameters. Based on the approximate IDA curves, the median seismic capacity at the ultimate limit state and the dispersion measures for demand and capacity are estimated and compared to the analysis case, in which the epistemic uncertainties are neglected. In addition, the results are also compared with the results of the N2 method [12], which was for the case of the example building previously applied in the companion study [13].

Two limit states were defined for comparison reasons between the results of the approximate IDA and results of the N2 method. The definition of the limit states differs with respect to the methods involved since the N2 method is not capable to predict global dynamic instability of the structure. In the case of the approximate IDA the limit state was therefore defined with the global stability of the structure (on the following, termed as ultimate limit state), whereas in the case of the N2 method, the limit state under consideration corresponded to the displacement at 80 % of its maximum base shear in relation to the softening part of the pushover curve (termed as near-collapse (NC) limit state). More about both limit states is explained within the related sections, for example, in section 3.4.

3.1 Description of the structure and mathematical model

The example structure is a three-storey RC frame building, which was designed for the earthquake resistance according to the Eurocode 8 [11]. The plan dimensions of the structure are 9.7×10.5 m. All three storeys are 3 m high, with monolithic slabs having a depth of 15 cm. The strength of the concrete and of the steel reinforcement amounted to 33 and 400 MPa, respectively. The masses and corresponding mass moments of inertia amounted to 94.3 t and 1667 tm^2 for the first two storeys, and 94.4 t and 1634 tm^2 for the top storey, respectively. The plan view of the building and the reinforcement of typical cross sections of the columns and beams are shown in Figure 2.

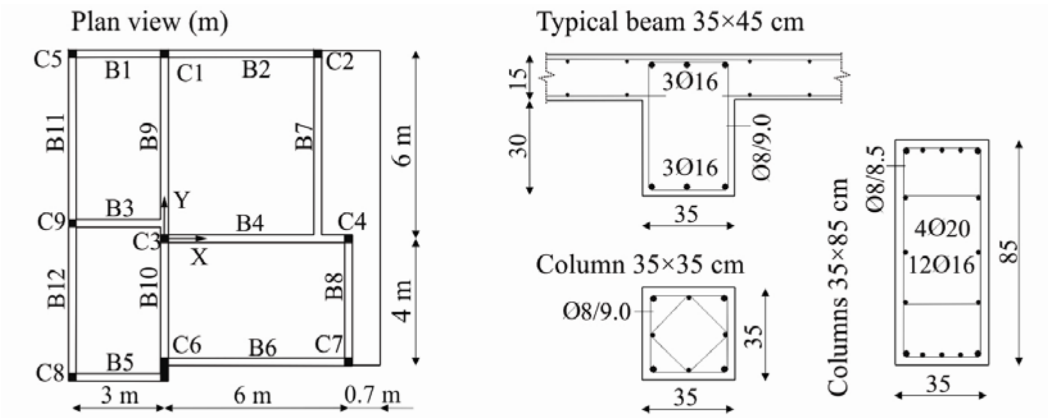

Figure 2: The plan view of the building and typical cross-sections of the columns and beams.

The structure was modelled by one-component lumped plasticity elements, which consist of an elastic beam-column element and two inelastic rotational hinges at the ends. Envelopes, describing a moment-rotation relationships of the hinges, were modelled with equivalent tri-linear relationship using effective initial stiffness of the elements. The three characteristic points of the envelopes are the yield point (Y), the maximum strength (M), and a near-collapse point (NC). The NC point represents a local near-collapse limit state in a column or beam, which is defined by the ultimate rotation θ_u, corresponding to a 20 % reduction in the maximum moment. Near-collapse rotation in the columns was estimated by means of the Conditional Average Estimator (CAE) method [14], whereas in the beams the near-collapse rotation for their hinges was determined by using the formula defined in the Eurocode 8-3 [15].

In the structural model, the rigid diaphragms were assumed at the floor levels due to the monolithic RC slabs. Consequently the masses were lumped at the mass centres. Centreline dimensions of the elements were used with the exception of beams B5 and B6 (Figure 2). These beams are connected eccentrically to column C6.

All analyses were performed with the OpenSees [16] in conjunction with the PBEE toolbox [17].

3.2 Input random variables and the set of structural models

In order to incorporate the epistemic uncertainty into the structural model, the following parameters were treated as random variables: the strength of the concrete and of the reinforcement steel, mass, effective slab width and the parameters describing the characteristic rotations in the plastic hinges, i.e. the yield and the ultimate plastic rotation. These model parameters were defined with the eight random variables, each representing individual model parameter. Note that the random variables, except for those defining the characteristic rotations in the plastic hinges, were assumed to be uncorrelated. In the study, the correlation factor between the yield and the ultimate rotation in plastic hinges was assumed to be 0.5. The statistical parameters were taken from literature and are presented in Table 1.

Utilizing the Latin Hypercube Sampling (LHS) technique [9], the selected random variables were used to generate the set of 20 structural models (simulations) in addition to the deterministic structural model in order to incorporate the selected sources of the epistemic uncertainties. Note, that the number of simulations generated is slightly higher than two times the number of input random variables. This is about the smallest reasonable number of the simulations, if using the LHS technique. A smaller number of simulations would probably lead to a significant decrease in the accuracy of the results. For more detailed discussion on the effects of number of the simulations on the results of the seismic performance evaluation when using the LHS technique, the reader is referred to the previous study [5].

Variable	COV	Distribution	Reference
Concrete strength	0.20	Normal	Melchers [18]
Steel strength	0.05	Lognormal	JCSS [19]
Mass	0.10	Normal	Ellingwood [20]
Eff. Slab width	0.20	Normal	Haselton [21]
Yield rotation:			
• columns	0.36	Lognormal	Panagiotakos and Fardis [22]
• beams	0.36	Lognormal	
Ultimate rotation:			
• columns	0.40	Lognormal	Peruš et al. [14]
• beams	0.60	Lognormal	Panagiotakos and Fardis [22]

Table 1: The statistical characteristics of the input random variables.

3.3 Results of the pushover analysis

The results of the pushover analysis are, for the simplicity reasons, presented only for the pushover analysis in positive X direction (Figure 3). The horizontal load pattern for pushover analysis was determined by the product of storey masses and the first modal shape in X direction.

The pushover curves for the deterministic structural model are compared with pushover curves of the set of 20 structural models incorporating the epistemic uncertainties (Figure 3). In addition, the 16th, 50th and 84th fractiles of the base shear given the top displacement (“fractile pushover curve”) are also shown in Figure 3. A high scatter in global ductility of the structure is observed. For example, the comparison between the fractile pushover curves and the deterministic pushover curve shows that the epistemic uncertainties mostly affect the displacement ductility of the structure. The epistemic uncertainties have moderate effect on the maximum base shear, while the difference in the initial stiffness between different structural models is almost negligible.

A large difference between the pushover curves presented in Figure 3 can be attributed to the fact that different collapse mechanisms were observed for different structural models, some of them being illustrated in Figure 4, for example at the NC limit state. The most influential parameter, which governs the type of the near-collapse mechanism of the building, is the rotation capacity of the beams. In the case, if rotation capacity of the beams is low in comparison to that of the columns, then the strength degradation of structure is a consequence of the damage in beams of the 1st and 2nd storeys. Thus, the top displacement capacity at NC limit state is low D_t = 0.18 m (Figure 4a). The opposite case, i.e. the damage in plastic hinges of the structural model with high rotation capacities of the beams, is presented in Figure 4b. In this case, the top displacement capacity at NC limit state is equal to 0.59 m.

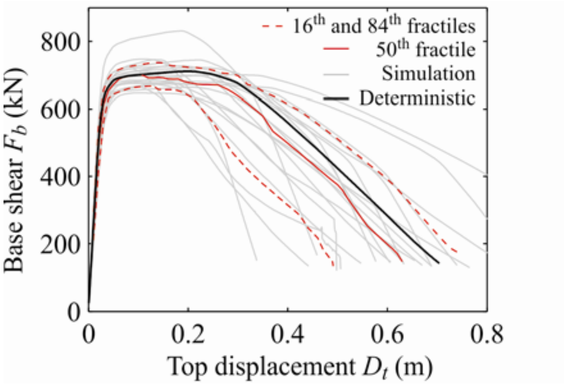

Figure 3: The pushover curves for the deterministic structural model, for the set of 20 structural models and the fractile pushover curves, representing the 16th, 50th and 84th fractiles of the base shear given the top displacement.

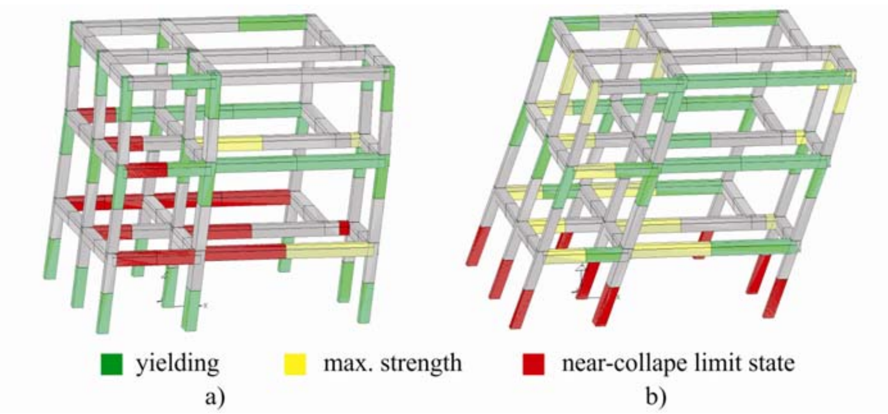

Figure 4: The damage in the plastic hinges of the building for a) the model with the smallest displacement capacity at the NC limit state, and b) for the model with the highest displacement capacity at the NC limit state. The results presented are based on the pushover analysis.

3.4 Prediction of IDA curves and definition of the ultimate limit state

The approximate IDA curves of the structure were determined by using a web application, which involves response database of the equivalent SDOF model. For that reason, the pushover curves were, for the need of the presented case study, idealized with a four-linear force-displacement relationship as presented in Figure 5. The first two points of the idealized force-displacement relationship represent the cracking (Cr) and yielding (Y) of the idealized system, the third point represents the maximum deformation capacity at the end of the plastic plateau (U), and the forth point represents a complete loss of the base shear capacity. The cracking point was defined in the elastic part of the pushover curve and corresponded to the top displacement of the structure at which the first element (column or beam) starts yielding. The yield force F_y was assumed equal to the maximum base shear obtained by the pushover analysis. The post-cracking stiffness of the idealized system was determined in such a way that the areas under the pushover curve and the idealized force-displacement relationship were equal (termed as an equal energy rule) taking into account the interval between displacement at the cracking of concrete D_{cr} and displacement at the maximum base shear D_m. Similar rule applied also in determining the third point of the force-displacement relationship. The difference between pushover curve and the idealized force-displacement relationship is small as presented in Figure 5 for the deterministic structural model.

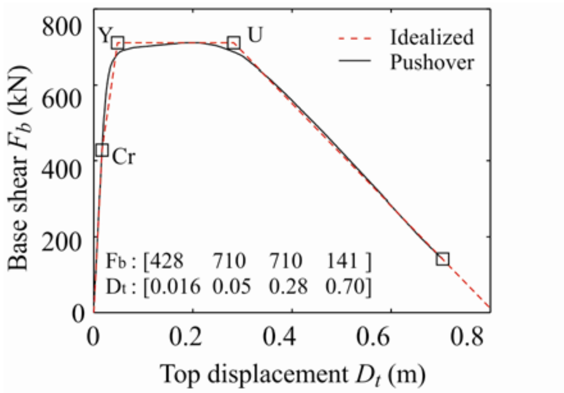

Figure 5: The pushover curve of the deterministic structural model and the corresponding idealization using a four-linear force-displacement relationship.

The modification factor Γ which relate the spectral displacement of an equivalent SDOF model to the roof displacement of the MDOF system is defined as follows:

$$\Gamma = \frac{m_{SDOF}}{\sum m_i \cdot \phi_i^2}, \; m_{SDOF} = \sum m_i \cdot \phi_i, \tag{4}$$

where m_{SDOF} is equivalent mass of the SDOF model, m_i is the mass of the structure in the i-th storey and ϕ_i is the displacement in the i-th storey, normalized to the displacement at the top storey. The vector ϕ was taken equal to the first modal shape of the structure, which corresponds to translations in X direction. The additional two parameters, which also affect the dynamic response of the SDOF model, are the critical damping factor (ξ), which was in our study assumed equal to 5 %, and the period of the equivalent SDOF model (T_{SDOF}) calculated as:

$$T_{SDOF} = 2 \cdot \pi \cdot \sqrt{\frac{m_{SDOF} \cdot d_{cr}^*}{f_{cr}^*}}, \tag{5}$$

where d_{cr}^* and f_{cr}^* are, respectively, the displacement and the base shear at cracking of the equivalent SDOF model.

Using the procedure described above, the equivalent SDOF models were determined for the deterministic structural model and for 20 structural models incorporating the epistemic uncertainties. Then, by applying the web application, the IDA curves of each structural model have been approximately predicted based on the dynamic analysis of the corresponding equivalent SDOF models. Actually, the output of the approximate IDA, which is presented in the Figure 6, are the IDA points, calculated for different ground motion records and for multiple levels of seismic intensities, and the fractile IDA curves, i.e. the median IDA curve and associated 16^{th} and 84^{th} fractiles for the top displacements.

One of the objectives of this study was estimation of the seismic performance parameters, i.e. the median seismic capacity and the corresponding dispersion measures. These parameters have been in the case of the approximate IDA estimated for the ultimate limit state of the structure, which has been, in our case, defined with the global loss of the structural stability in relation to the approximate IDA curves. These points of the ultimate limit state are marked red in the Figure 6.

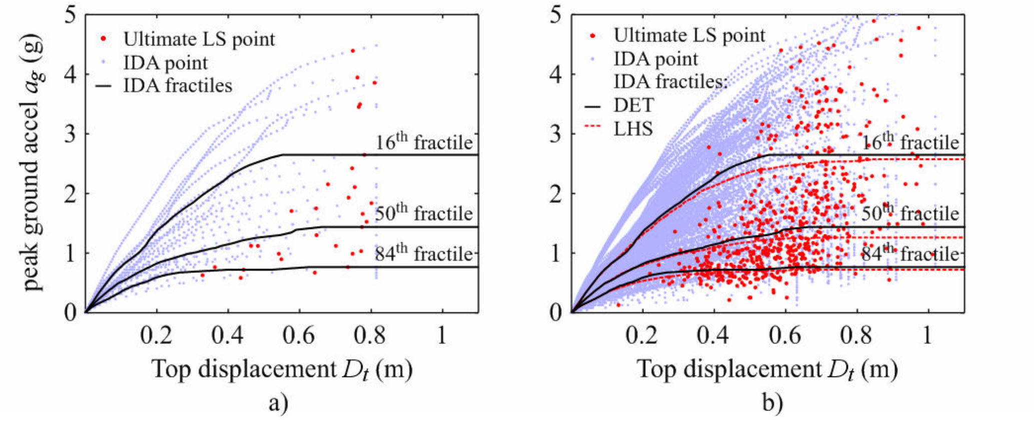

Figure 6: The fractile approximate IDA curves, collapse points and approximate IDA points for a) approximate IDA based on the deterministic structural model and b) for the approximate IDA based on the set of structural models representing the epistemic uncertainties.

Note, that such definition of the ultimate limit state is not completely realistic and probably overestimates the displacement capacity, since in the model the linear post-capping stiffness is assumed all the way to a complete loss of the base shear. Therefore the displacement, which corresponds to the global dynamic instability of structure, is probably smaller than that, which corresponds to the highlighted points in the Figure 6.

In the case if seismic response parameters were determined with the N2 method [12], which was for the same structure used in one of the previous studies [13], the displacement corresponding to the global dynamic stability of structure cannot be obtained. Usually it is conservatively assumed that this displacement corresponds to the NC limit state, as already defined above.

Nevertheless, the difference in the estimated seismic capacity, which was made due to the different definition of the limit states, is small, if expressed with the difference of predicted peak ground acceleration that cause the ultimate limit state or the NC limit-state. This is so, since the gradient of peak ground acceleration in comparison to the top displacement is low in the range of the displacements near global collapse of the structure.

By comparing the results in Figures 6a and 6b it can be concluded that a high scatter in the seismic performance of the structure arises predominantly due to the aleatory uncertainties. For example, the 16th and 84th fractiles of the approximated IDA curves for the deterministic structural model (see Figure 6a), for which only the aleatory uncertainty due to the randomness in the ground motion records were incorporated, and corresponding 16th and 84th fractiles of the approximated IDA curves incorporating both the aleatory and the epistemic uncertainties (see Figure 6b), do not differ significantly. This is observed especially for the 84th fractile and for the first part of the 16th fractile IDA curve. That means that the greatest part of the scatter in seismic performance is due to the random nature of the seismic load, whereas the epistemic uncertainties are relatively minor contributor to overall performance uncertainty. Another fact that can be concluded based on the results presented in Figure 6 is that the median seismic performance at the region of the intensities for the ultimate limit state is reduced if the epistemic uncertainties are included into the analysis. The reduction of the median seismic performance at the ultimate limit state can be visually observed as a vertical shift in the prediction of the median (50th fractile) IDA curve.

3.5 The estimation of seismic response parameters

The maximum top displacement $D_{t,C}$ at the defined ultimate limit state and corresponding peak ground acceleration $a_{g,C}$ were selected for seismic response parameters and assessed with its median value and dispersion measure. In addition, dispersion in top displacement demand was also estimated since it represents an input parameter for the seismic risk assessment if evaluated utilizing the EDP-based formulation. Note that in this case the dispersion depends on the peak ground acceleration and was for the sake of simplicity computed at a_g = 0.4 g.

Another distinction between the seismic response parameters was made with respect to the type of the uncertainty, which were considered in the analysis. So, the seismic response parameters were estimated, respectively, for the deterministic structural model, where only the aleatory uncertainties due to the record-to-record variability (R) were considered, and for the stochastic structural model, where both the aleatory and epistemic uncertainties (RU) were simultaneously incorporated. In addition, the results were presented also for the case if seismic response parameters were estimated by using the N2 method. In this case, the results were provided for the deterministic structural model and for the stochastic structural model with consideration of epistemic uncertainties (U).

In all cases the median and the dispersion measure of the seismic response parameters were estimated by using three different methods, all assuming a log-normal distribution of sample values. Thus the dispersion measure was defined as the standard deviation of natural logarithms. The first considered method was a so called counting method (on the following noted as M1), since 16th, 50th and 84th fractiles (y_{16}, y_{50}, y_{84}) of the seismic response parameters were determined according to the counting method and the corresponding dispersion measure was calculated as the average value of $\beta_{16} = \log(y_{50}/y_{16})$ and $\beta_{84} = \log(y_{84}/y_{50})$. The method of moments was the second method (M2) used for assessing median $\tilde{m}$ and dispersion measure β of the seismic response parameters, i.e. the 1st and the 2nd moment of the sample values:

$$\tilde{m} = \bar{m} \cdot e^{-0.5\cdot\beta^2}, \tag{6}$$

$$\beta = \sqrt{\ln\left(\frac{\sigma^2}{\bar{m}^2}+1\right)}, \tag{7}$$

where ($\bar{m}$) is the average value and (σ) is the corresponding standard deviation of sample values. Lastly, the third method (M3) was the maximum likelihood method, where median and corresponding dispersion were estimated as follows:

$$\tilde{m} = e^{\frac{1}{N}\sum_{i=1}^{N}\log(y_i)}, \quad \bar{m}_{\log Y} = \log\left(\tilde{m}\right), \tag{8}$$

$$\beta = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(\log(y_i) - \bar{m}_{\log Y}\right)^2}, \tag{9}$$

where y_i is the i-th sample value and N is size of the sample.

The results for estimated median and dispersion of the seismic response parameters with respect to the three adopted methods are presented in Tables 2 and 3.The notations DET and LHS relate, respectively, to the results based on deterministic and the stochastic structural model.

It can be observed (Table 2) that the median peak ground accelerations ($\tilde{a}_{g,C}$) are very similar if estimated with the N2 method and the approximate IDA. The average value of $\tilde{a}_{g,C}$ for deterministic structural model estimated by the approximate IDA equals to 1.43 g, which is only around 6 % higher to that obtained with the N2 method ($\tilde{a}_{g,C}$ = 1.35 g). The estimated $\tilde{a}_{g,C}$ is larger if compared to that, which was obtained in the study conducted by Rozman and Fajfar [23] ($\tilde{a}_{g,NC}$ = 0.77 g), since adopted top displacement at the near collapse limit state was significantly smaller than that defined for the case of this study.

A slightly higher difference between the results of the approximate IDA and the N2 method presented in Tale 2 can be observed for the case if epistemic uncertainties are incorporated in the analysis. In that case, the average value of $\tilde{a}_{g,C}$ is reduced and amounts to 1.25 and 1.14 g if estimated by using the approximate IDA and the N2 method, respectively. However, in the contrast to the median peak ground acceleration, the predicted median displacement capacity strongly depends on the way how the limit state is defined. For example, the estimated median displacement capacity for deterministic structural model varies, respectively, from 0.39 m to 0.64 m (by the average), if calculated for the NC limit state as adopted for the N2 method, or for the ultimate limit state as adopted for the approximate IDA. Note that probably none of these values is not correct but represent more or less the boundaries in the prediction of the ultimate displacement capacity of the example structure, suggesting that the displacement at the ultimate limit state as defined in the case of the approximate IDA most probably overestimates the structural displacement capacity due to the adopted structural model, whereas the displacement capacity at NC limit state as defined in the N2 method is rather a conservative estimate for the displacement capacity.

Another phenomenon, which can be observed by comparing the results in the Table 2, is that the median seismic capacity is typically reduced if epistemic uncertainties are considered in the analysis. Thus neglecting the epistemic uncertainties leads to unsafe prediction of median capacity. In the case of the approximated IDA, the median peak ground acceleration at the ultimate limit state is, by the average, around 12 % less than that, which was estimated in the case of the deterministic model. Similar, conclusion can be made based on the results of the N2 method. A relatively high shift in the prediction of median seismic response parameters for the ultimate limit state cannot be simply ignored.

Method		$\tilde{a}_{g,C}(g)$	Δ	$\tilde{D}_{t,C}(m)$	Δ
DET + N2		1.35	-	0.39	-
LHS + N2	M1	1.15	-15 %	0.36	-8 %
	M2	1.08	-20 %	0.31	-21 %
	M3	1.20	-11 %	0.35	-10 %
DET + app. IDA	M1	1.48	-	0.71	-
	M2	1.33		0.59	
	M3	1.49		0.64	
LHS + app. IDA	M1	1.26	-15 %	0.61	-20 %
	M2	1.19	-11 %	0.53	-10 %
	M3	1.32	-11 %	0.58	-9 %

Table 2: The median seismic response parameters ($\tilde{a}_{g,C}$ and $\tilde{D}_{t,C}$) estimated by using the approximate IDA and the N2 method. The relative contribution of the epistemic uncertainties to the reduction of the median capacity is also shown and calculated as Δ = (LHS - DET) / DET.

Important results of this study are the dispersion measures in seismic demand and capacity. Based on the results presented in Table 3, the highest dispersion can be observed for the peak ground accelerations at the ultimate limit state. For example, the average β_{agCR} is 0.57 if only the results based on the second and the third statistical methods (M2 and M3) are considered.

The results of the counting method are excluded, since, it provides overestimated value for β_{agCR} if compared to that determined by the method of moment or maximum likelihood method.

Much smaller dispersion is observed for the displacement corresponding to the ultimate limit state. In this case, by the average, the dispersion in top displacement for the aleatory uncertainties amounts to around 0.24. However, the latter dispersion is slightly increased to the value of 0.26 if the epistemic uncertainties are incorporated into the model. This is true also for the IM-based response parameter. In particular, the dispersion for the peak ground acceleration at the ultimate limit state due to the aleatory and epistemic uncertainties β_{agCRU} increased, by the average, from 0.57 to 0.59. It is a very small increase in the dispersion. For the presented example can be therefore concluded that that epistemic uncertainties have in the contrast to the median estimates of seismic capacity almost negligible influence to the overall dispersion in structural seismic performance.

The dispersion measures in top displacement demand have been calculated for the peak ground acceleration 0.4 g, which approximately corresponds to the seismic intensity at which the 84th fractile IDA curve, if determined with consideration of epistemic uncertainties, becomes horizontal. However, by the average, the dispersion in the displacement demand due to the aleatory uncertainties β_{DR} amounts to 0.44. The dispersion in displacement demand due to aleaotry and epistemic uncertainty β_{DRU} cannot be determined by utilizing the method of moments or the maximum likelihood method since, for some cases, the collapse was attained at a_g=0.4 g. Therefore, β_{DRU} was estimated by using the counting method and equals to 0.49.

In addition, the Table 3 represents also the dispersion measures due to the epistemic uncertainties, which have been determined in the previous study [13] by applying the N2 method. These values amount to around 0.30 and 0.31, respectively, for the peak ground acceleration and the top displacement at the NC limit state and can be used to calculate the total dispersion in structural seismic performance by using the square root of the sum of the squares of the dispersions for aleatory and epistemic uncertainties.

Method		Peak ground accel. ($a_{g,C}$)		Displ. Capacity (C)		Displ. Demand (D)	
LHS + N2	M1	β_{agCU}	0.27	β_{CU}	0.33	β_{DU}	0.11
	M2		0.30		0.30		0.09
	M3		0.31		0.31		0.09
DET + app. IDA	M1	β_{agCR}	0.72	β_{CR}	0.25	β_{DR}	0.46
	M2		0.56		0.21		0.44
	M3		0.58		0.25		0.43
LHS + app. IDA	M1	β_{agCRU}	0.64	β_{CRU}	0.26	β_{DRU}	0.49
	M2		0.58		0.24		-
	M3		0.60		0.27		-

Table 3: The dispersion measures due to the aleatory uncertainty (randomness) $(\beta_{agCR}, \beta_{CR}, \beta_{DR})$ from approximate IDA, the dispersions due to aleatory and epistemic uncertainty $(\beta_{agCRU}, \beta_{CRU}, \beta_{DRU})$ determined based on the approximate IDA, and the dispersion measures due to epistemic uncertainty $(\beta_{agCU}, \beta_{CU}, \beta_{DU})$ determined by the N2 method.

4 CONCLUSIONS

In the presented study, a methodology for fast seismic performance assessment was presented. The procedure involves approximate IDA by using the web application. The effects of the epistemic uncertainties are considered by the set of structural models, which are determined by utilizing the Latin Hypercube Sampling (LHS) technique. The outcomes of this process are the seismic response parameters in terms of the median seismic capacity and corresponding dispersion measures, which can be estimated considering both the aleatory and epistemic uncertainties.

The use of the presented methodology was illustrated by applying it to the case of three storey RC frame building. For the example structure it was shown that the aleatory and epistemic uncertainties strongly affect the overall dispersion in seismic performance of structure. It was also shown that neglecting the epistemic uncertainty potentially leads to unsafe design, since median seismic capacity at ultimate limits state can be overestimated if epistemic uncertainties are neglected in the analysis. For example, the median seismic capacity is reduced for about 11 - 15 % if epistemic uncertainties are incorporated into the approximate IDA.

All the analyses have been performed by using the PBEE toolbox in conjunction with the OpenSees. It was proved as a very powerful toll for fast seismic performance assessment of structures, since it allows performing a numerous number of the required computational simulations enables quick post-processing of the results and damage visualization on the structure for different limit state and for different structural models.

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