Probabilistic assessment of the seismic damage in reinforced concrete buildings

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Summary

The main objective of this article is to assess the expected seismic damage in reinforced concrete buildings from a probabilistic point of view by using Monte Carlo simulation. To do that, the seismic behavior of the building is studied by using random capacity obtained by considering the mechanical properties of the materials as random variables. Starting from the capacity curves, one can obtain the damage states and the fragility curves as well as to develop curves describing the expected seismic damage of the structures as a function of a seismic hazard characteristic. The latter can be calculated using the capacity spectrum and the demand spectrum according to the methodology proposed by the RISK-UE project. For defining the seismic demand as a random variable, a set of real accelerograms are obtained from the European and Spanish databases in such a way that the mean of their elastic response spectra is similar to an elastic response spectrum selected from Eurocode 8. In order to combine the uncertainties associated with the seismic action and the mechanical properties of materials, two procedures are considered for obtaining functions which relates the PGA to the maximum spectral displacements. The first one is based on a series of nonlinear dynamic analyses. The second one is based on the well known procedure named equal displacement approximation exposed in ATC 40. After applying both procedures, the probability density functions of the maximum displacement at the roof of the building are obtained and compared. The expected structural damage is finally obtained by replacing the spectral displacement obtained by using the ATC 40 and the incremental dynamic procedure. In the damage functions the results obtained from incremental static and dynamic analyses are finally compared and discussed from a probabilistic point of view.

KEYWORDS: Probabilistic seismic assessment, capacity curves, vulnerability, fragility curves, expected damage.

1. INTRODUCTION

The vulnerability of structures subjected to earthquakes can be evaluated numerically either by using incremental static analysis or pushover analysis or by means of nonlinear dynamic analysis performed in an incremental way. All the variables involved in such structural analyses, mainly the mechanical properties and the seismic actions, should be considered as random. The reason is that the randomness of the implied variables combined with the uncertainties in the seismic hazard, may lead to an underestimation or an overestimation of the actual vulnerability of the structure, but they are not always treated in this way. Due to the current capacity of the computers, a great number of structural analyses can be performed in order to study the behavior of buildings from a probabilistic standpoint within the framework of Monte Carlo simulation.

This study focuses on the nonlinear seismic response of reinforced concrete buildings and on their damage analysis considering the involved uncertainties (Fragiadakis & Vamvatsikos 2009). In the pushover analysis, previous studies already considered uncertainties (Bommer & Crowley 2006; Borzi et al. 2007; Fragiadakis & Vamvatsikos 2010) and evaluated the nonlinear behavior of structures considering uncertainties in the mechanical properties of materials and in the nonlinear static analysis (pushover) by means of the Monte Carlo method. Dolsek (2010) considered in this type of studies the seismic action as a random signal using real accelerograms roughly compatible with design spectra, but did not take into account the uncertainties associated to the structural characteristics. The present paper aims to assess the seismic vulnerability of the structure considering both the mechanical properties of the materials as random variables and the seismic actions as random signals. The seismic demand for the area is obtained in probabilistic terms starting from the response spectrum chosen from Eurocode 8. Afterwards, a procedure for selecting accelerograms whose response spectra are compatible, in a mean sense, with the mentioned response spectrum, is applied. In this study, we compare the results carried out by using the above mentioned analyses: 1) Incremental static analysis or pushover analysis (PA). 2) Nonlinear dynamic analysis (NLDA) in an incremental way, that is, an incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002). PA and NLDA have been compared in previous studies (Mwafy & Elnashai 2001; Poursha et al. 2007; Kim & Kuruma 2008). The PA is used to determine the capacity curves of the structure and to obtain the expected displacement at the roof of the building for a given seismic area (Borzi et al. 2008; Barbat et al. 2008; Lantada et al. 2009; Pujades et al. 2011). The roof displacement obtained with this procedure will be considered as a random variable and will be compared with the displacement calculated via IDA. Finally, the results are discussed and compared from a probabilistic point of view.

2. DESCRIPTION OF THE STUDIED BUILDING

The reinforced concrete building selected for this study is shown in Figure 1. The building is located in Spain and, therefore, some of the selected accelerograms are taken from the Spanish database. However, due to the low seismicity of the area, we use additional accelerograms taken from the European database. The building is regular in plan, allowing the use of a 2D model. Nevertheless, this building has not a framed structure but a structure with columns and slabs which, in this case, are waffled slabs. In Spain, this type of building is frequently used for family housing and for offices and has been previously studied (Vielma et al. 2009; Vielma et al. 2010). For the purpose of this study, we use a simplified equivalent framed model. The main geometric characteristics of the building can be seen in Figure 1.

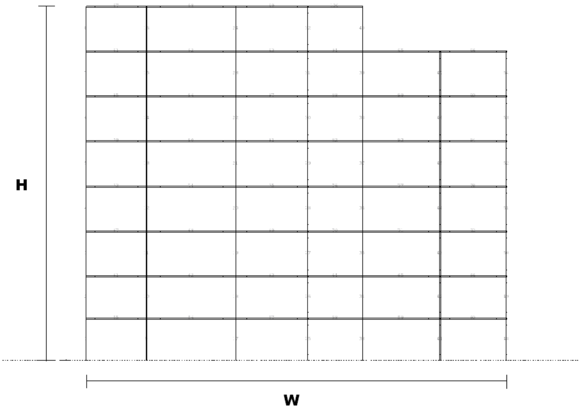

Figure 1. Equivalent frame of the reinforced concrete structure used in the probabilistic simulation

The constitutive law of the structural elements is elastoplastic without hardening or softening. In order to define the yield surfaces for the material of the columns and beams, it is necessary to create an interaction diagrams between the bending moment and the axial force, and between the bending moment and the angular deformation, respectively. Programs have been developed in MATLAB in order to calculate the yielding points which are necessary when defining the behavior of the structural elements used in the nonlinear static and in the dynamic analyses of the structures which, in this article, are performed by means of the RUAUMOKO computer software (Carr 2000).

3. MONTECARLO SIMULATION

3.1. Nonlinear static analysis

As mentioned before, the mechanical properties of the materials, such as the concrete compressive strength, *fc*, and the reinforced yield strength, *fy*, are random variables. The distribution assumed for these variables is Gaussian; the parameters that define these distributions, the mean value μ and the standard deviation σ, as well as the coefficient of variation r, are shown in Table 1. These parameters correspond to the values given in the original blueprints of the structure.

Table 1. Characteristics of the probability distribution of the mechanical properties of the structural elements

	Col 1	Col 2	Col ..
fc	30000	1000	3.33%
fy	411510	22093	5.36%

It is important to note that, in each pushover analysis, the strength of the structural elements is not constant because, for each of them, a new stochastic data is generated. On the other hand, in the PA the results change depending on the variation of the load pattern with the height. Besides, it is very difficult to establish how much to increase the load; moreover, a load maintaining the pattern corresponding to the first mode of vibration of the elastic structure cannot capture the effect of higher-modes (Poursha et al. 2008). To overcome these difficulties, we use the so-called adaptive pushover method in its version proposed by Satyarno (1999) and it is this one which will be referred in the following as PA. The advantage of the PA is the independency of the results on the loading pattern, because this is calculated as a function of the mass, the equivalent frequency and the deformed shape of the structure; furthermore, the horizontal load limit is controlled by the current stiffness of the structure. Figure 2 shows a comparison among different capacity curves calculated for different load patterns.

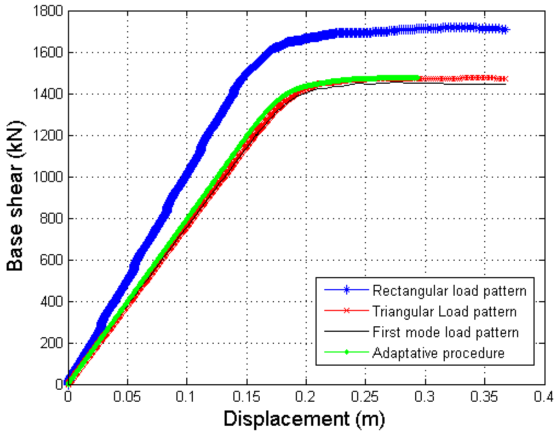

Figure 2. Capacity curves obtained with different load patterns

After generating 1000 samples of mechanical properties *fc* and *fy* by using the latin hypecube method, 1000 capacity curves are obtained and plotted in Figure 3 in which the uncertainties in the results can be seen.

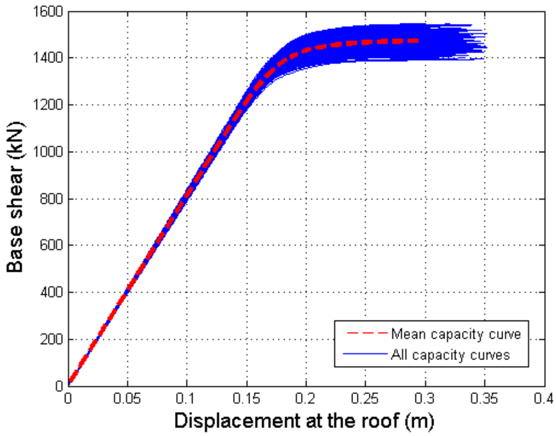

Figure 3. Capacity curves obtained via Monte Carlo simulation

3.1. Nonlinear static analysis

In order to consider the randomness of the seismic action, the response spectrum corresponding to EUROCODE 8 of type 1 and for soil type D is taken as target. Although we performed several tests using the type 2 spectra, we eventually used in this article the type 1 spectrum for soil D, in order to achieve the nonlinear inelastic behavior of the structure, because for type 2 spectra the accelerograms require to be scaled for peak ground accelerations (PGA) higher than those expected in Spain; in the following, we will refer to this spectrum as the code spectrum. 20 acceleration records are selected whose mean 5% damped elastic response spectrum is in the range of +/- 5% of the code spectrum. There are several methods for selecting the accelerograms which describe the seismic hazard of an area (Hancock et al. 2008). In this paper we use a procedure based on least squares which consists in selecting a group of accelerograms whose mean spectrum minimize the error respecting the target spectrum (Vargas et al. 2012). In Figure 4, the code spectrum and the mean spectrum corresponding to the 20 selected accelerograms are shown.

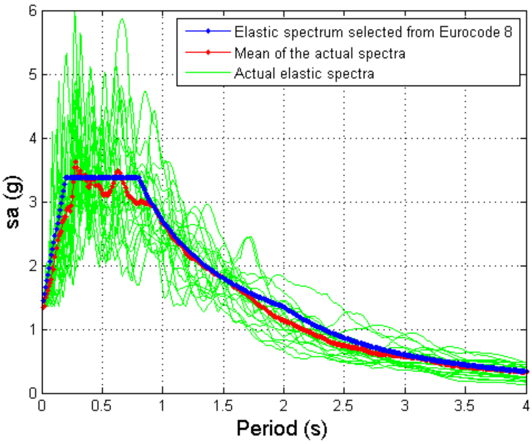

Figure 4. Comparison between the spectrum of EUROCODE 8 and the mean of the spectra of the earthquakes selected from the Spanish and European databases

The selected accelerograms are scaled to different levels of the peak ground acceleration and are then used to perform a series of NLDA within the framework of the incremental dynamic analysis, IDA. The scaling method used in this article consists of incrementing the acceleration ordinates by a scalar allowing to define the desired PGA levels. Even if in this way we maintain the initial frequency content of the seismic action, this scaling method is adequate for the purpose of this article, which is the comparison in a probabilistic way of the results obtained with static and dynamic nonlinear analysis methods considering uncertainties. Nevertheless, in the Monte Carlo simulation we considered 20 accelerograms scaled in this way, having their frequency content certain variability. The role of IDA in this article is to combine the uncertainties in the mechanical properties of the building with those involved in the seismic action. The objective is to obtain the evolution of a dynamic response variable, for instance the displacement at the roof of the building, as a function of a variable describing the seismic action; we considered that this last variable is the PGA, which, in this case, is increased up to 0.25 g which is the maximum PGA value in the Spanish seismic design code. In the IDA, the variable which is related to the PGA is the expected spectral displacement (ESD). Obviously, as the seismic demand is obtained as a random variable, the ESD will also be random and, therefore, the values shown in Figure 5 are the mean values. This figure shows the variation of ESD, when PGA increases, together with the +/-1.65 standard deviation intervals that is, a confidence level of 95%. Figure 6 shows the evolution of the standard deviation of the ESD.

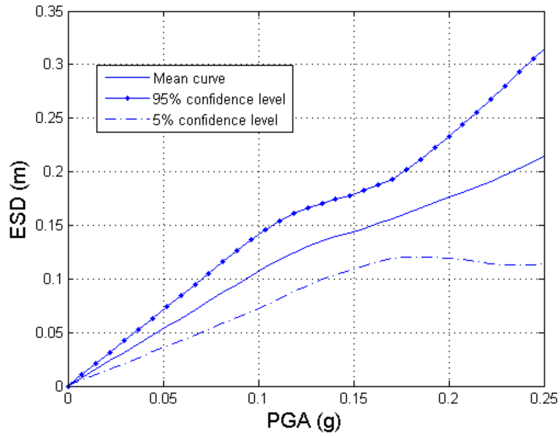

Figure 5. Relation between PGA and mean expected spectral displacement

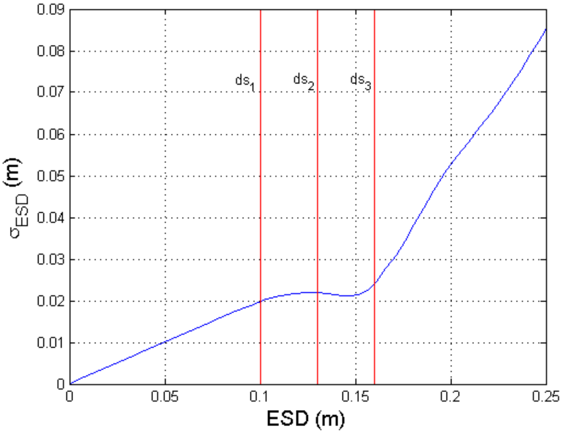

Figure 6. Relation between the mean expected spectral displacement and the standard deviation

In Figure 5, a major change in the slope of the curve approximately for 0.16 g can be seen, but other significant changes in the behavior of the structure can be not appreciated in this graph. However, Figure 6 shows three points related to significant changes in the slope of the curve; these points can be also seen in Table 2, where the first is related to the first change of the slope of the curve. The second point corresponds to the first maximum and the third one is related to the beginning of a straight line with maximum slope. Later on, we will use these points to discuss the damage states of the structure and, thus, its seismic behavior.

Table 2. Coordinates of the particular points which are identified in Figure 5

Point	X coordinates (m)	Y coordinates (m)
1	0.1	0.02
2	0.13	0.0294
3	0.16	0.0298

4. CAPACITY SPECTRUM, DAMAGE STATES AND FRAGILITY CURVES

4.1. Capacity spectrum and bilinear representation

Once calculated the capacity curve of the structure, it is useful transforming it in the capacity spectrum by means of the procedure proposed in the ATC-40 (1996). The capacity spectrum is represented in spectral acceleration-spectral displacement coordinates (*sa*-*sd*) and is often used in its simplified bilinear form, defined by the yielding point (*Dy*, *Ay*) and the ultimate capacity point (*Du*, *Au*), as it can be seen in Figure 7.

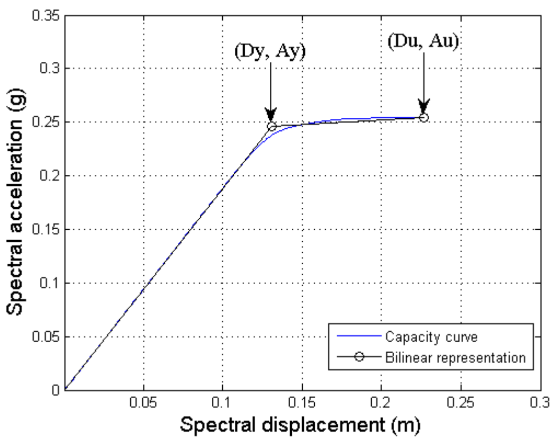

Figure 7. One of the capacity spectra of the studied building and its bilinear representation

4.2. Damage states

In order to analyze the expected damage we use simplified methods allowing to obtain the damage states thresholds *ds* and the corresponding fragility curves. Four non-null damage states are considered: (1) *slight*, (2) *moderate*, (3) *severe* and (4) *extensive-to-collapse*. For a given damage state, according to the hypothesis considered in the RISK-UE project (Milutinovic & Trendafiloski 2003), the damage state threshold is defined by the 50% probability of occurrence. This damage state threshold can be defined in the following simplified way from the bilinear capacity spectrum (Lantada et al 2008; Barbat et al 2010; Barbat et al 2011):

$$\begin{gathered} ds_1 = 0.7 * Dy \\ ds_2 = Dy \\ ds_3 = Ds_2 + 0.25 * (Ds_4 - Ds_2) \\ ds_4 = Du \end{gathered} \tag{1}$$

The damage states thresholds have been established for all the capacity spectra calculated for the studied structure. Thus, considering the damage states thresholds as random variables, Figure 8 shows the results obtained and the mean values for each damage state. This figure also shows how the dispersion increases when the damage states increase. This fact indicates that, when the structure enters into nonlinear behavior, the uncertainties in the damage level increase. The mean and standard deviation of *sa* and *sd* of each damage state are shown in Table 3. It is important noting the agreement of these values with those of Table 2. The points 1, 2 and 3 of Table 2 correspond to the mean values of the damage state thresholds ds_1, ds_2 and ds_3; the changes in the slope of the standard deviation calculated via IDA correspond to the damage state thresholds indicated in Figure 6.

Table 3. Mean and standard deviation of damage states.

ds	m_{sd} (m)	s_{sd} (m)	m_{sa} (g)	s_{sa} (g)
1	0.0985	0.0013	0.1878	0.0038
2	0.1314	0.0017	0.2504	0.0050
3	0.1583	0.0049	0.1878	0.0051
4	0.2212	0.0148	0.2504	0.0054

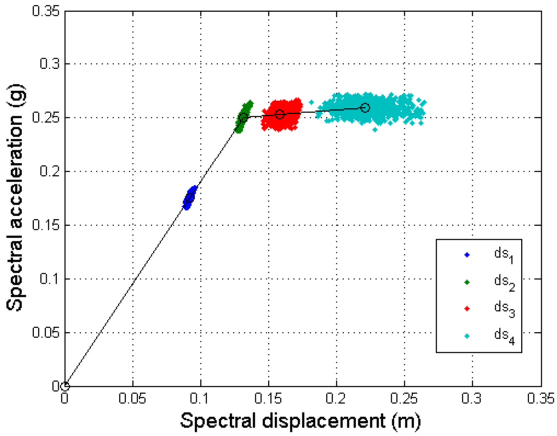

Figure 8. Damage states as random variables

For each damage state threshold, the corresponding fragility curve is defined by the probability of being exceeded the corresponding threshold as a function, in our case, of the spectral displacement. It is assumed that the fragility curves follow a standard lognormal cumulative distribution function.

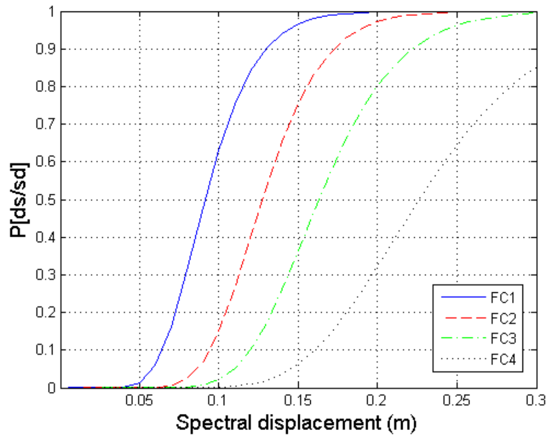

Figure 9. Example of one fragility curves for the building

Each fragility curve is then obtained by using the following equation:

$$P[i / sd] = f\left[\frac{1}{b_{ds_i}} Ln\left(\frac{sd}{\overline{\overline{sd}}_{ds_i}}\right)\right] \tag{2}$$

where *sd* is the spectral displacement and $\overline{sd}_{ds_i}$ is the mean value of the lognormal distribution which is the corresponding damage state threshold as defined above. b_{ds} is the standard deviation of the natural logarithm of the spectral displacement of ds. In equation 2, the mean values, $\overline{sd}_{ds_i}$, can be determined from the capacity spectrum and b_{ds} can be estimated by assuming that the damage follows a binomial distributions and, finally, by using a mean square procedure to fit the fragility curves (see Lantada et al. 2008). Notwithstanding, there is a correlation between the ductility of the building and the variables b_{ds} of each fragility curve, which has been found by relating the results obtained with the Monte Carlo Method. This correlation is very useful because one can obtain the fragility curves applying directly this method, avoiding the mean square procedure described in Lantada et al. 2008, being thus the calculation time considerably reduced. Figure 10 shows graphically this correlation.

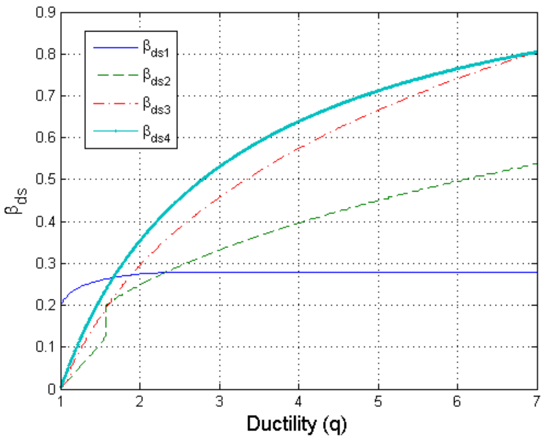

Figure 10. Correlation between ductility and the variables b_{ds}

Figure 11 shows 1000 fragility curves obtained for all the calculated capacity spectra applying the simplified method exposed above. Obviously, according to Figure 8, as the considered damage state increases, the uncertainties involved in the corresponding fragility curve also increase. As an example, the fragility curves corresponding to the damage sates *slight* and *collapse* can be seen in Figure 11. Figure 12 shows the results of a sensitivity test on the influence of the mechanical properties of the materials and the damage state thresholds; the stiffness is used as an independent variable in this test.

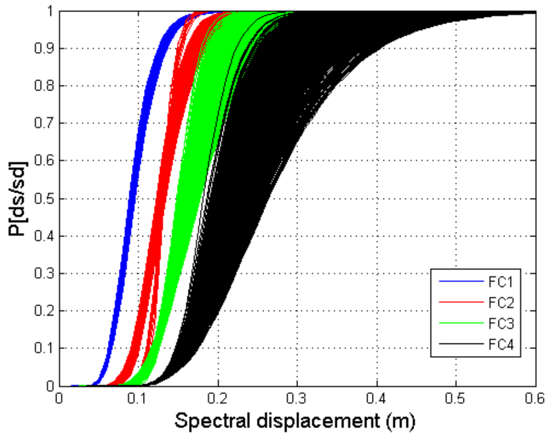

Figure 11. Fragility curves as random variables

ds_1 and ds_2 damage states are practically independent on stiffness, while for ds_3 and ds_4 the spectral displacement decreases with increasing stiffness, indicating that the probability of the corresponding damage states increases with stiffness. Figure 13 shows the mean fragility curves and Figure 14 shows the corresponding standard deviations as a function of *sd*.

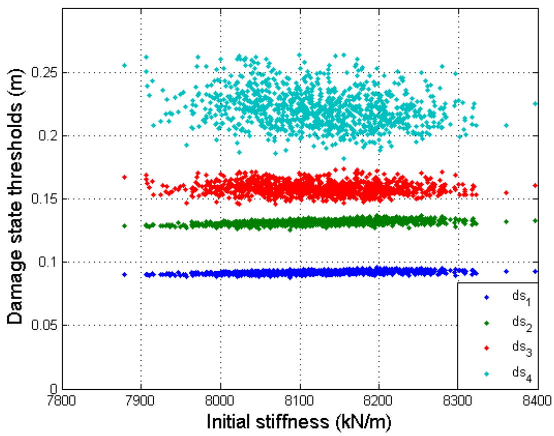

Figure 12. Sensitivity test for *ds* and the initial stiffness

Figure 13 clearly depicts the dependence of the uncertainties on the damage grades. For instance, the coefficient of variation of the damage state *ds4* may be greater than 10%, which means that, for a confidence level of 95%, the increase in the probability of failure will be greater than 16.5%. This increase reaffirms the importance of analyzing the problem from a probabilistic viewpoint.

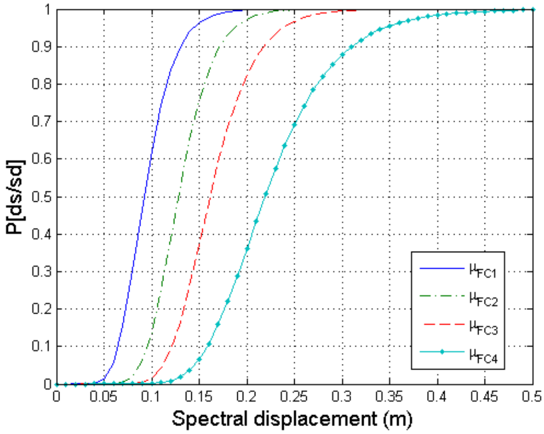

Figure 13. Mean fragility curves obtained via Monte Carlo simulation

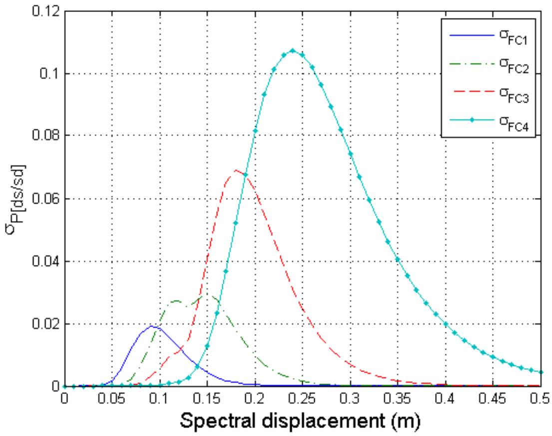

Figure 14. Standard deviation of the fragility curves

5. EXPECTED SPECTRAL DISPLACEMENT AND DAMAGE INDEX

The maximum expected spectral displacement in a building due to the seismic hazard of the area is obtained in section 4 using NLDA and the results were presented in figures 5 and 6. Different studies have searched for simplified procedures to estimate the expected spectral displacement (Kim et al 2008). A much more simplified procedure is the so-called equal displacement approximation, EDA, which is described in ATC-40 (1996) (see also Mahaney 1993). The EDA is performed by using the spectra corresponding to the selected accelerograms in order to perform a better comparison with the results obtained from the NLDA. Due to the fact that the EDA is a linear procedure, the results will be linear; for this reason, it is enough to scale the spectra for a single PGA. In order to express the expected spectral displacement as a function of the PGA, the spectra are scaled to 0.25 g obtaining the mean and standard deviation. Figure 15 shows graphically the EDA procedure considering the uncertainties associated to the seismic action and to the mechanical properties of the materials.

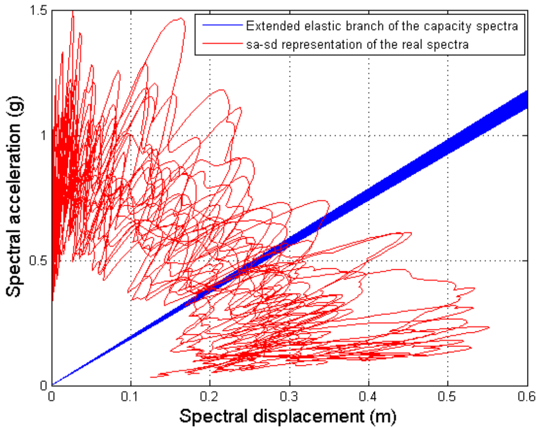

Figure 15. Equal displacement approximation considering the uncertainties associated to the seismic action and to the mechanical properties of the materials

These results are shown in figures 16 and 17, respectively, where the NLDA results are also given. The main conclusion of this analysis is that the EDA methodology provides an adequate approximation for the expected spectral displacement of the building, because it does not underestimate the expected displacement.

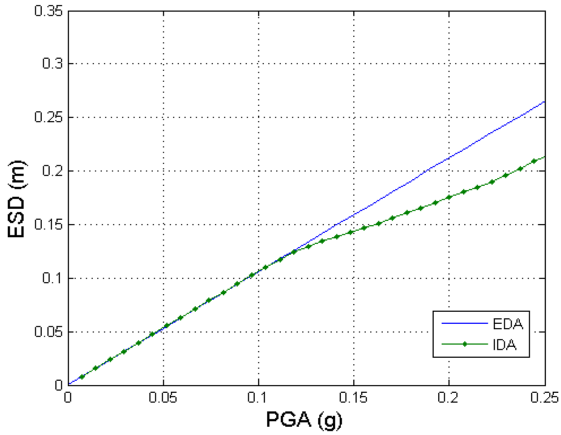

Figure 16. Comparison between the relation of the PGA to the expected spectral displacement obtained by using EDA and NLDA

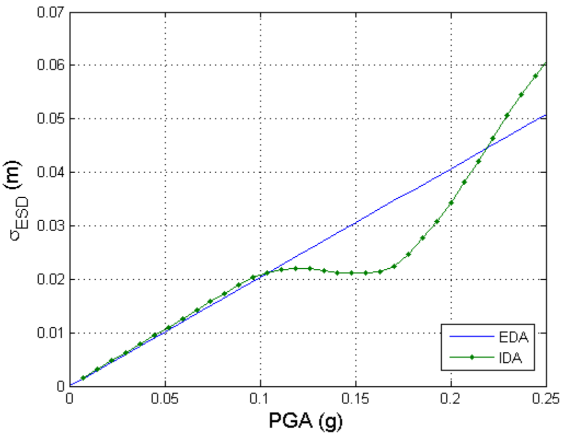

Figure 17. Comparison between the relations of the PGA to the standard deviation of the expected spectral displacement obtained by means of EDA and NLDA

Moreover, from a probabilistic viewpoint, this method is also conservative because, in the nonlinear range, the standard deviation obtained with EDA is higher than that obtained with NLDA. On the other hand, we can calculate a damage index, *DI*, which is defined by the following equation:

$$DI = \frac{1}{n}\sum_{i=0}^{n} iP(ds_i) \tag{3}$$

where *n* is the number of non-null damage states (*n*=4 in this case) and *P*(*dsi*) is the probability of the damage state *i* which can be easily calculated from the fragility curves (see Figure 18). *DI* is the normalized mean damage grade which is a measure of the overall damage in the structure (Barbat et al. 2008). The authors proposed equation (3) for calculating the overall damage taking into account that the higher damage states ds_i have more influence on the global damage state *DI* of the structure and, also, because this equation provides the main parameter of the binomial distribution which allows obtaining the fragility curves in a simpler manner. Obviously, the values of the coefficients of the four probabilities of the damage states (0.25, 0.5, 0.75, 1.0) could be calibrated in order to improve *DI* if observed damage values would be available.

DI can be also plotted as a function of the expected spectral displacement. Thus, *DI* can be calculated for any spectral displacement but, in order to include the randomness associated to the seismic action, the comparison between *DI* obtained with EDA and with NLDA requires computing the PGA corresponding to each spectral displacement by using the relation shown in Figure 16. Figure 19 shows the obtained results, namely the mean values and the 95% confidence level curves. Again, our results confirm that the EDA is conservative respect to NLDA, even when considering a confidence level of 95% for random variables. But, if the variables were not treated by using a probabilistic approach, this would result in an underestimation of the actual damage that may occur in the building. In the case of the building analyzed in this article, the damage index estimated by using a deterministic approach is 0.25 lesser than that computed from a probabilistic viewpoint.

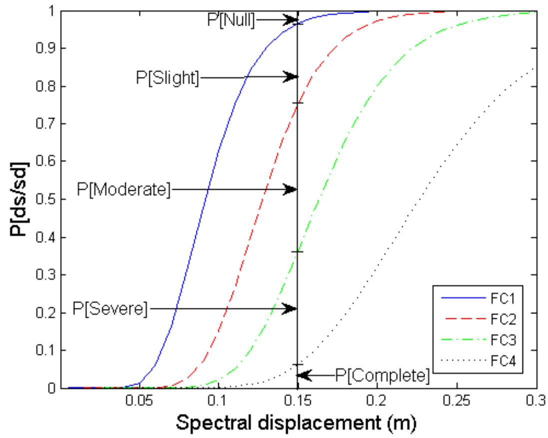

Figure 18. Probability of each damage state depending on the expected spectral displacement

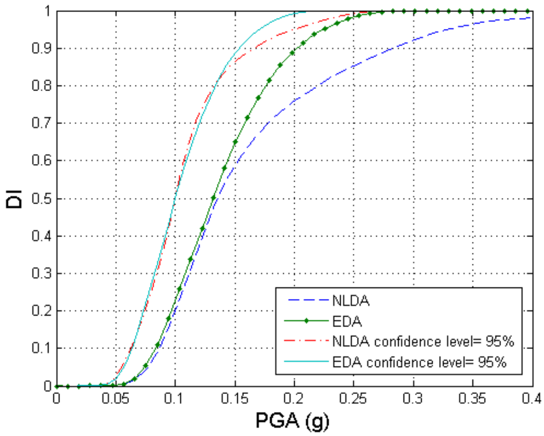

Figure 19. Damage index obtained with NLDA, EDA, and a confidence level of 95%

6. DISCUSION AND CONCLUSION

The assessment of the vulnerability, fragility and expected damage in a reinforced concrete building is performed in this article. However, the results obtained herein go further and they compare, in a probabilistic way, nonlinear static and dynamic analysis procedures. We face the problem from a probabilistic point of view, since we consider the uncertainties in the parameters related to the mechanical properties of the materials and the seismic demands. A first hint is that, notwithstanding that incremental dynamic analysis is a powerful tool in the assessment of the structural behavior of buildings when submitted to seismic actions, this procedure has little sense if the seismic demand is not carefully and properly selected. We put special care in the selection of the accelerograms used in this study. We have selected accelerograms corresponding to seismic events from the Spanish and European strong motion records databases. In order to reach a wide range of spectral displacements, the Eurocode type 1 design spectrum for soil type D has been taken as target demand. The accelerograms have been selected according to this criterion and have been scales to cover PGA values until 0.25 g. We used standard pushover analysis to obtain probabilistic capacity curves. A modified adaptive technique has been used to define the horizontal incremental load limit in order to stop automatically the pushover analysis during the run of high number of structures, 1000 in this case. Starting from the capacity spectra, simplified methods allow obtaining damage states thresholds and probabilistic fragility curves. An interesting conclusion of this exercise is that uncertainties increase in the nonlinear range. For the *collapse* damage state, the uncertainties in the fragility curves may be greater than 10%. EDA and NLDA are used to obtain the expected spectral displacement and its standard deviation as a function of the PGA. Again, uncertainties increase with increasing PGA. This fact can be attributed to the increase of the inelastic behavior of the building. EDA is a successful approach because it does not underestimate the actual displacement, but it can be too conservative in structures with higher ductility. Furthermore, the fact that both the expected spectral displacement and the standard deviation are greater when calculated with EDA than when calculated with NLDA, confirms that EDA is conservative. In the NLDA, the seismic action is the main responsible for uncertainties in the spectral response, being less significant the influence of the uncertainties in the mechanical properties of the building. However, as the damage state increases, a sensitivity test shows a correlation between stiffness and spectral displacement. For the damage states ds_3 and ds_4, the spectral displacement decreases when stiffness increases, indicating that the probability of the corresponding damage state increases with the stiffness. This result is important since the damage states ds_3 and ds_4 have a high influence upon the calculation of the damage index. Finally, the comparison of the damage index as a function of PGA and the corresponding uncertainties shows that, for damage states from *severe* to *collapse* and for a confidence level of 95%, the uncertainties in the damage index may be higher than 0.25 units or 42% of the damage index. Thus, perhaps, the most important conclusion is that both static and dynamic structural analyses should be faced by using probabilistic approaches.

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