

INCREMENTAL DYNAMIC ANALYSIS AND PUSHOVER ANALYSIS. A PROBABILISTIC COMPARISON

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Abstract. *Capacity-spectrum-based-methods are also used for assessing the vulnerability and risk of existing buildings. Capacity curves are usually obtained by means of nonlinear static analysis. Incremental Dynamic Analysis is another powerful tool based on nonlinear dynamic analysis. This method is similar to the pushover analysis as the input is incrementally increased but it is different as it is based on dynamic analysis. Moreover, it is well known that the uncertainties associated to the structural response can be significant, because the uncertainties involved in the mechanical properties of the materials and the expected seismic actions are also highly uncertain. In this work selected mechanical properties are considered as random variables and the seismic hazard is considered in a probabilistic way. A number of accelerograms of actual European seismic events have been selected in such a way that their response spectra fitted well the response spectra provided by the seismic codes for the zone where the target building is constructed. In this work a fully probabilistic approach is tackled by means of Monte Carlo simulation and it is applied to a detailed study of the seismic response of a reinforced concrete building. The building is representative for office buildings in Spain but the methods used and the results obtained can be extended to other types of buildings. The main purposes of this work are 1) to analyze the differences when static and dynamic techniques are used and 2) to obtain a measure of the uncertainties involved in the assessment of the vulnerability of structures. The results show that static based procedures are somehow conservative and that uncertainties increase with the severity of the seismic actions and with the damage. Low damage state fragility curves have little uncertainty while high damage grades fragility curves show great scattering.*

1 INTRODUCTION

Aiming to prevent the seismic risk, it is necessary to assess the vulnerability of existing structures. To do that, several methods have been proposed, starting from different approaches. One is the vulnerability index method in which the action is defined from the EMS-98 by macroseismic intensities and structural behaviour through a vulnerability index [1, 2]. Another highly used method is based on the capacity spectrum. In this, the seismic action is defined by means of the elastic response spectra and the vulnerability or fragility of the building by means of the capacity curve; the latter is calculated from an incremental nonlinear static analysis, commonly known as "Pushover Analysis" [3, 4 5]. Another tool used to evaluate the performance of structures against seismic actions is the Incremental Dynamic Analysis (IDA) proposed by Vamvatsikos & Cornell [6]. The purpose of IDA is to obtain a measure of damage in the structure by increasing the intensity of the action record, in this case the peak ground acceleration. Vamvatsikos & Cornell make an interesting analogy between the PA and the IDA, because both procedures increases the load on the structure and measure the response of the system in terms of a control variable which may be the maximum displacement at the roof, the maximum inter storey drift, etc. This procedure allows obtaining the dynamic response of a structure when the seismic action is increased. On the other hand, the mechanical properties of the materials which constitute the structure and the seismic action are random variables and, therefore, the vulnerability of the building is also a random variable. To take into account the inherent randomness of the problem, it is appropriate to use the Monte Carlo method. Therefore, in this paper, a probabilistic comparison between the PA and the IDA is performed when calculating the vulnerability of an existing reinforced concrete building. The main conclusion from this comparison highlights the importance of measuring the vulnerability of structures taking into account that the variables involved are random. This approach, mixed with powerful tools to analyze the structure such as the PA and the IDA, provide valuable information that can hardly be obtained with other methodologies.

2 BUILDING DESCRIPTION

This paper analyzes a reinforced concrete structure, consisting of columns and waffle slabs, which is part of the North Campus of the Universidad Politècnica de Catalunya. It has 7 levels and 4 spans, the height is 24.35 m and the width is 22.05 m (see Figure 1). The fundamental period of the building is 0.97 seconds. This value is higher when compared to that of conventional reinforced concrete buildings, because in the numerical model, the waffle slabs are approximated with beams of equivalent inertia and, therefore, are structural elements wide and flat leading to a reduction of the lateral stiffness of the structure. In the calculation model, the structural elements (equivalent beams and columns) follow an elastic-plastic constitutive law, which does not take into account either hardening or softening. Yielding surfaces are defined by the moment-axial load interaction diagram in columns and by the moment-angular deformation interaction diagram in beams.

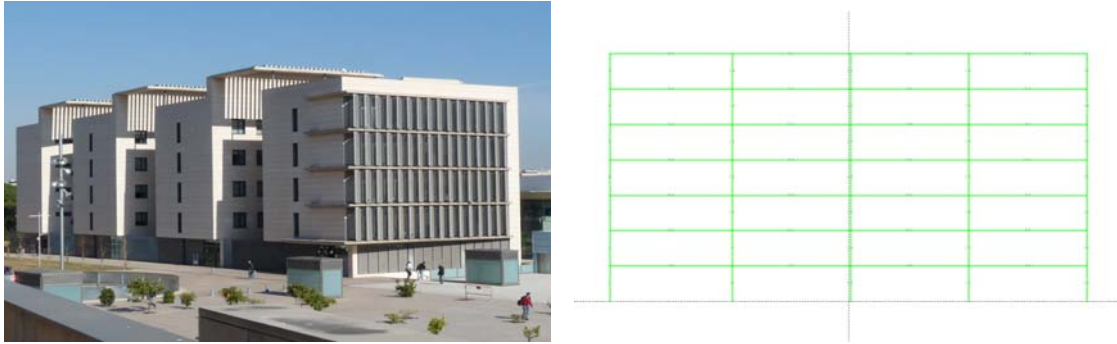


Figure 1. Picture of the building omega located in the Universidad Politécnic de Cataluña, Barcelona, Spain.

3 DAMAGE INDEX BASED ON PUSHOVER ANALYSIS

A tool often used to evaluate the behaviour of the structures against seismic loads, is the nonlinear static analysis, commonly called Pushover Analysis (PA). This numerical tool consists in apply a horizontal load to the structure, according to a certain pattern of forces, and in increasing its value until the structural collapse is reached. From this procedure one obtain a relationship between the displacement at the roof of the building and the base shear, called capacity curve. In this article, due to the probabilistic approach, the PA is performed repeatedly, therefore, it is appropriate to apply a procedure for obtaining automatically the horizontal load limit. For this, Satyarno [7] proposes the adaptive incremental nonlinear analysis that establishes the horizontal load limit as a function of the tangent fundamental frequency, i.e. the frequency associated with the first vibration mode, which is being calculated for each load increment. Therefore, in each step is calculated the first mode of vibration to determine the shape of the load in height. A detailed description of this procedure is found in the manuals of the program Ruaumoko [8] used for calculating the static and dynamic nonlinear structural response. As mentioned in the introduction, the mechanical properties of materials are considered as random variables. To do this, the values used in the structural design for concrete compressive strength f_c , and the tensile strength associated with steel yield strength f_y , are treated as random variables and using the inversion method of the cumulative probability distribution curve are generate 1000 random samples of these variables. In this paper, it is assumed that the generated random variables follow a Gaussian probability function whose mean and standard deviations are shown in Table 1.

	Mean Value (kPa)	Standard deviation (kPa)	Coefficient of variation
f_c	25000	2500	0.1
f_y	500000	50000	0.1

Table 1. Features of the Gaussian random variables considered.

For the generated samples, the PA is performed 1000 times and the capacity curve is obtained as the random variable shown in Figure 2.

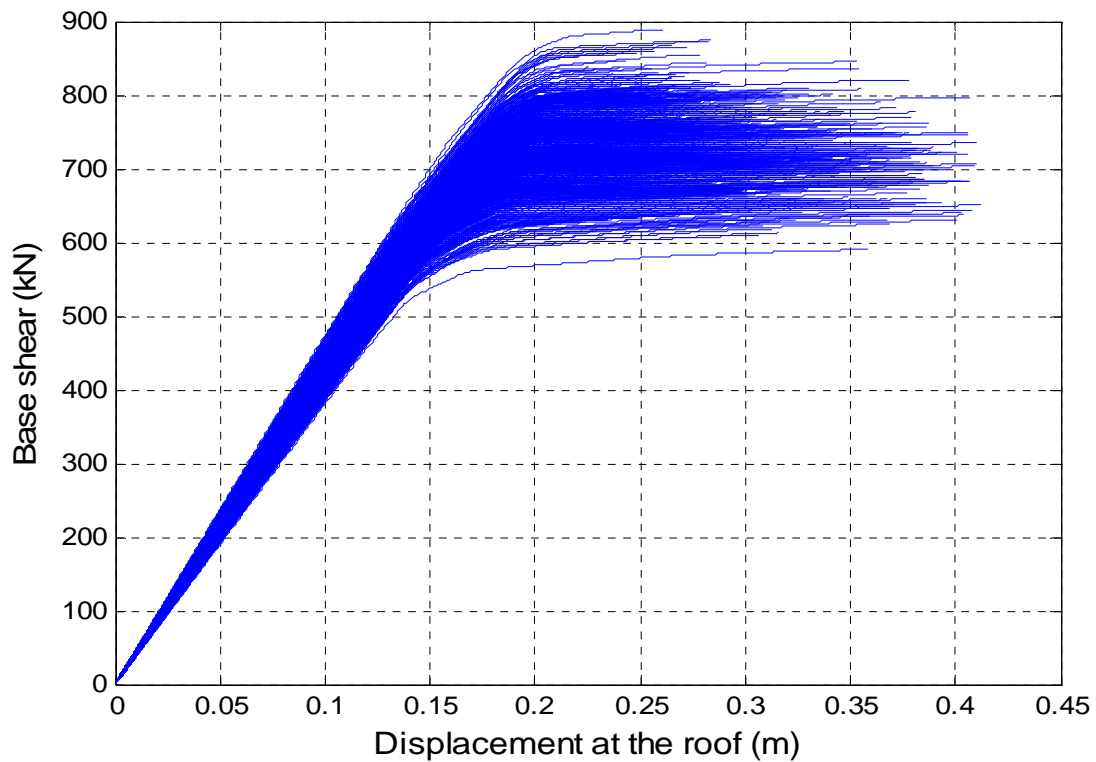


Figure 2. Capacity curves obtained from the PA, taking into account the uncertainty in the mechanical properties of materials.

The capacity curves shown in Figure 2 are transformed into capacity spectra, which relate the spectral displacement to spectral acceleration by means of the following equations [9]:

$$sd_i = \frac{\delta_i}{PF_1} \quad (1)$$

$$sa_i = \frac{V_i}{W/\alpha_i}$$

The subscript i in equations (1) is referred to the applied load increments on the structure during the PA; sd_i is the spectral displacement; δ_i is the displacement at the roof of the building; PF_1 is the modal participation factor of the first mode of vibration; sa_i is the spectral acceleration; V_i is the base shear; W is the weight of the building and α_i is the modal mass coefficient of the first mode of vibration. On the other hand, the capacity spectrum can be represented in a bilinear form, which is useful for defining damage states. Assumptions to build the bilinear capacity spectrum are: 1) The area under the bilinear curve must be equal to the area of the original curve. 2) The coordinates of the point of maximum displacement must be the same in both curves. 3) The slope of the initial branch should be equal in both curves. Figure 3 shows an example of the bilinear representation of the capacity spectrum. This can

be defined completely by the points (D_y, A_y) and (D_u, A_u) . These points are useful to define the states of damage, according to the procedure described in Lantada et al (2009).

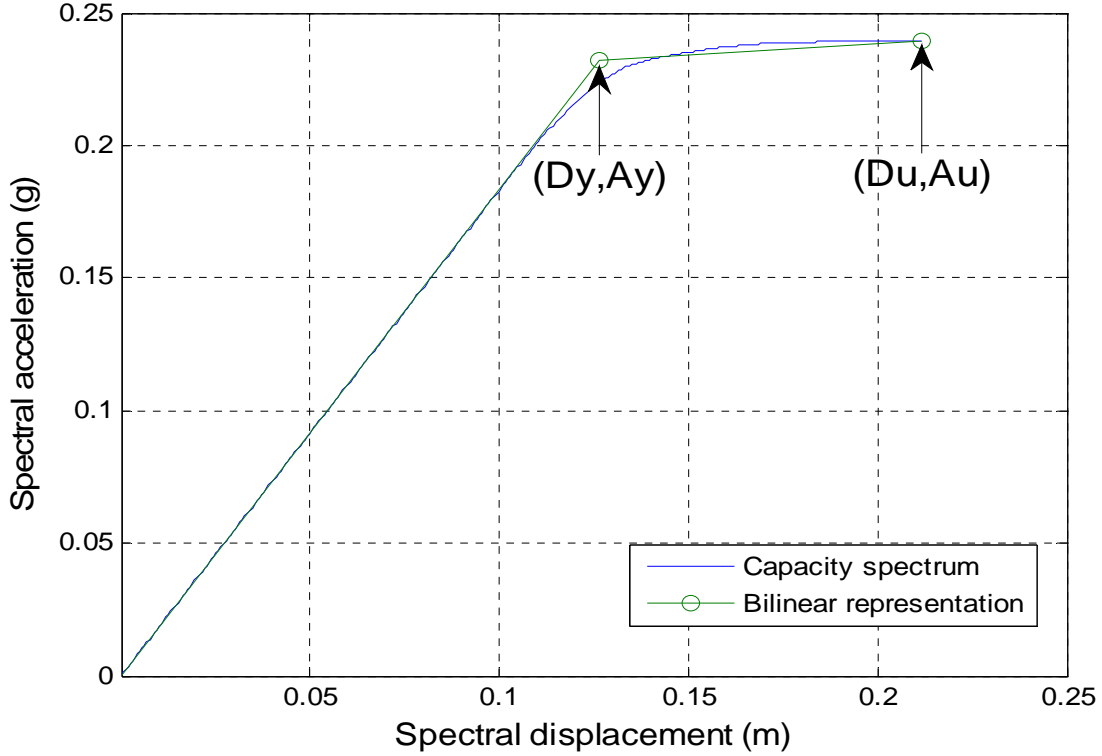


Figure 3. Capacity spectrum and the bilinear representation.

Different studies have been proposed to calculate the damage of the structure from the definition of damage states (ds), which are a description of the damage in the structure for a given spectral displacement. For example, HAZUS 99 [10] and Risk EU [11], define 4 ds , namely slight, moderate, extensive and complete. Description of the damage states depends on the type of structure. For example, According to HAZUS, in the case of reinforced concrete structures, the ds slight is described as the beginning of cracking due to bending moment or shear in beams and columns. Collapse state considers that the structure reaches an imminent risk of collapse. Risk EU seeks to define the damage states in simplified form, starting from the capacity spectrum in a bilinear representation. Based on the values (D_y, A_y) and (D_u, A_u) , the spectral displacements for the four damage states threshold ds_i are obtained according to the following equations:

$$\begin{aligned}
 ds_1 &= 0.7 * D_y \\
 ds_2 &= D_y \\
 ds_3 &= D_y + 0.25 * (D_u - D_y) \\
 ds_4 &= D_u
 \end{aligned}
 \tag{2}$$

Therefore, after calculating the capacity spectrum in bilinear representation and applying equations 2, it is possible to obtain the damage states thresholds as random variables, as shown in Figure 4.

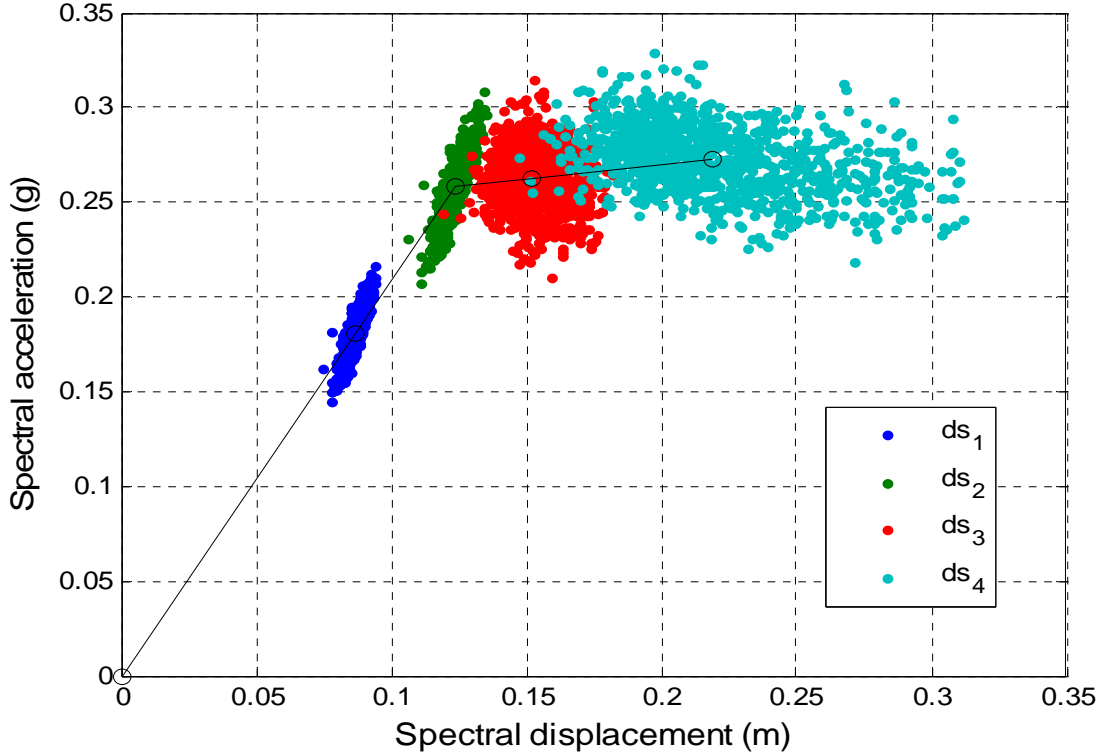


Figure 4. Damage states as random variables.

The mean, standard deviation and coefficient of variation of the damage states are shown in Table 2, in which it can be seen that the coefficient of variation of the damage state 4 is greater than that of the input variables. This is due to the fact that the problem is nonlinear and therefore it shows the importance of the probabilistic approach in this type of analysis.

	ds_1 (cm)	ds_2 (cm)	ds_3 (cm)	ds_4 (cm)
μ_{ds}	8.6	12.3	15.2	21.9
σ_{ds}	0.27	0.38	1.00	3.25
<i>c.v.</i>	0.03	0.03	0.06	0.15

Table 2. Mean value, standard deviation and coefficient of variation of the damage states.

After obtaining the damage states as random variables it is possible to calculate the fragility curves, which represent the probability of reaching or exceeding a damage state, in function of a parameter representing the seismic action. In this work, this parameter is the spectral displacement. To obtain the fragility curves the following assumptions must be considered: 1) The probability that the spectral displacements in each damage state threshold, ds_i , equals or

exceeds the damage state is 50%. 2) The fragility curves follow a lognormal cumulative probability function described by the following equation:

$$P[ds_i / sd] = \phi \left[\frac{1}{\beta_{ds_i}} \text{Ln} \left(\frac{sd}{ds_i} \right) \right] \quad (3)$$

where sd is the spectral displacement and β_{ds_i} is the standard deviation of natural logarithm of the variable ds_i . 3) The expected seismic damage in buildings follows a binomial probability distribution. Figure 5 shows all fragility curves calculated after applying the described procedure.

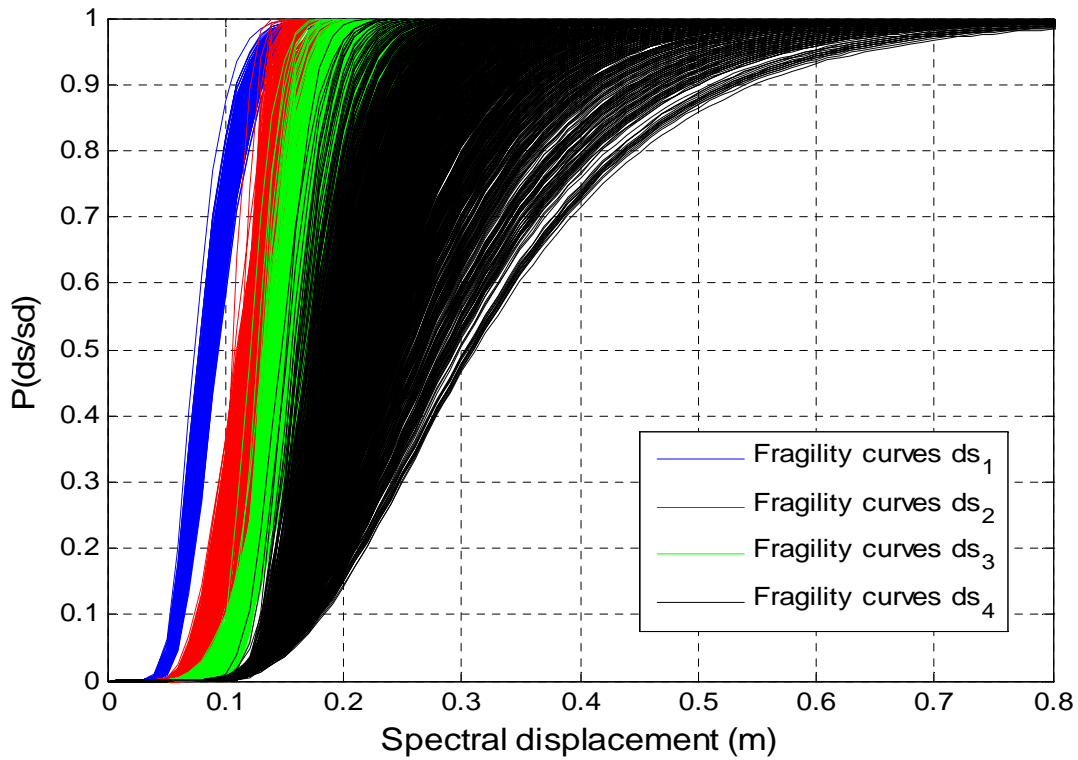


Figure 5. Fragility curves as random variables.

Since the probabilities of occurrence of each state of damage are easily obtained from the fragility curves, one can calculate the expected damage index, DI , which is the normalized mean damage state, which can be interpreted as a measure of the overall expected damage in the structure.

$$DI = \frac{1}{n} \sum_{i=0}^n iP(ED_i) \quad (4)$$

where n is the number of damage states considered, in this case 4 and $P(ds_i)$ is the probability of occurrence of ds_i . Figure 6 shows the ID calculated from the fragility curves of Figure 5. The curves of Figure 6 can be interpreted as random vulnerability curves.

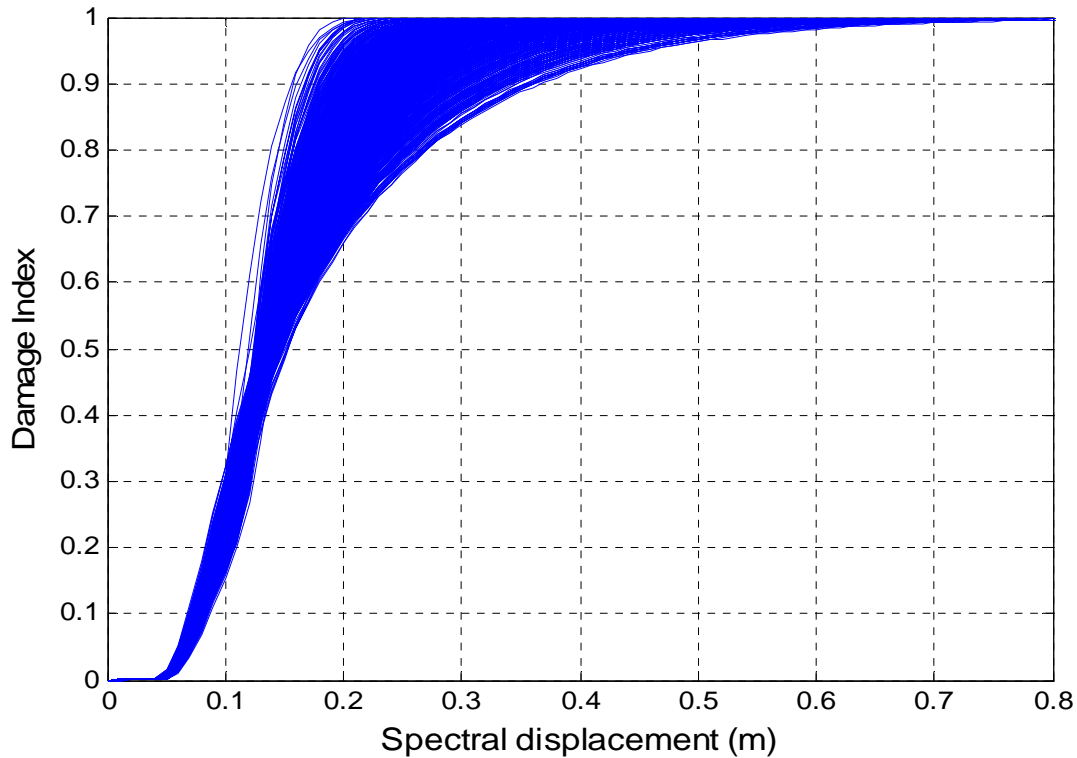


Figure 6. Damage index obtained starting from the PA as random variable.

4 DAMAGE INDEX BASED ON THE INCREMENTAL DYNAMIC ANALYSIS

Incremental dynamic analysis allows obtaining the dynamic response of a structure to an earthquake action. This earthquake is scaled to various PGA. As mentioned above, the purpose of this article is to compare the results obtained with the methodology based on the capacity spectrum with the incremental dynamic analysis. According to the probabilistic approach it is necessary to obtain the seismic action as a random variable. To do that, 20 earthquakes have been selected from two databases, one from Spain and the other from Europe, whose elastic response spectra are compatible with elastic response spectrum taken from Eurocode 8. In this case, the elastic spectrum type 1 and soil D is selected. Figure 9 shows the spectra of the selected earthquakes, their average value, and the spectrum type 1 soil D, taken from Eurocode 8.

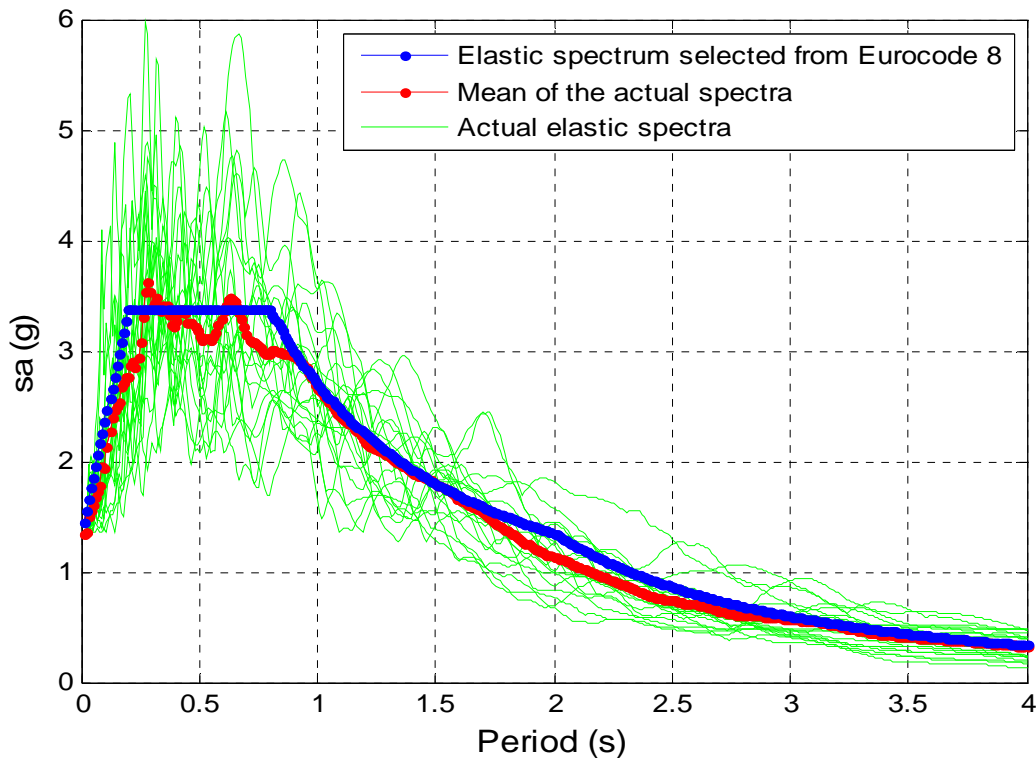


Figure 7. Selected spectra of the accelerograms that are compatible with spectrum type 1 soil D of Eurocode 8.

After selecting the accelerograms, the dynamic response of the structure is calculated, for different PGA until a maximum of 0.32 g, at intervals of 0.04 g. In each run of the nonlinear dynamic analysis, the damage index proposed by Park & Ang [12] is calculated and, also, the maximum displacement at the roof of the building, allowing to compare these results with those obtained previously from static procedure. Figure 8 shows the results obtained with both methods, and shows that the damage index obtained with the procedure based on the PA is conservative compared to the results obtained with the procedure based on IDA. However, when the damage index is close to 1, similar values are obtained with both procedures. On the other hand, it can be seen in the curves obtained with the PA procedure that the structural damage begins for a smaller spectral displacement than in the case of the IDA procedure and that, in both cases, slopes are similar. This means that the PA curves are shifted respecting the IDA curves what could be easily adjusted by changing the damage states coefficients. It is important to note the large scatter in both cases, showing the importance of assessing the vulnerability of structures from a probabilistic perspective, whichever procedure is used.

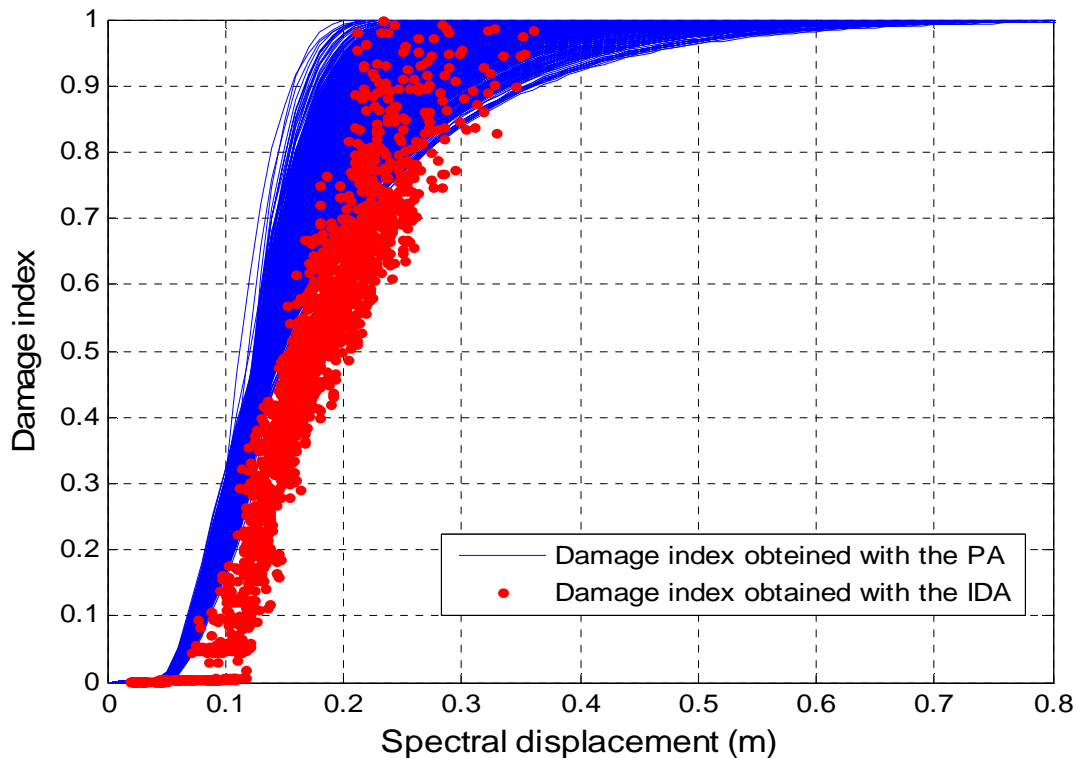


Figure 8. Damage index obtained with static and dynamic procedures.

5 CONCLUSIONS

In this work, the vulnerability of a real reinforced concrete structure, with columns and waffle slab has been assessed, taking into account that the input variables are random. Two approaches to evaluate the vulnerability of the building have been used. The first one is based on the pushover analysis and the second one is based on the incremental dynamic analysis. An important conclusion is that, despite working with advanced structural analysis, these procedures show significant uncertainties when taking into account the randomness of the variables associated with the problem. It should be emphasized that in this work relatively small coefficients of variation for input variables have been considered taking into account the uncertainties that may exist in older structures that did not have quality control and have not been designed according to the earthquake-resistant criteria. An important conclusion is that the results obtained with the procedure based on the capacity curve are conservative when compared with the results obtained with the incremental dynamic analysis. However, within the procedure based on the capacity curve, there are factors such as those given in equation 2, which can be modified to improve the correlation with the results based on dynamic calculation. The main conclusion of this paper is that whichever the procedure is used to evaluate the vulnerability of a structure, it is important to note that the input variables, such as the mechanical properties of materials and the seismic action, are random variables and these generate large uncertainties in the seismic response, which can lead to overestimate or to underestimate the real damage that can occur in a structure.

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