A Sampling Method For the Probabilistic Seismic Safety Assessment of Buildings

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SUMMARY:
The full characterization of a structure according to its seismic vulnerability is a major concern of the earthquake engineering community. In this context, the current scientific research trends are addressed to the seismic assessment of structures.
In this paper it is presented a Probabilistic Seismic Safety Assessment (PSSA) procedure that uses a sampling method, the Latin Hypercube, for computing the failure probability of reinforcement concrete buildings. The proposed methodology includes the main uncertainties of the structural problem, from both demand and capacity sides. The failure probability is, herein, obtained as the region of the points where the difference between the capacity and the demand is lower than the zero value, the collapse region.
A nonlinear static procedure is used to compute the seismic structural behaviour, comparing the results of the performance of buildings with the results obtained from nonlinear dynamic analyses. This comparison will try to verify the validity and accurateness of pushover analyses in a PSSA procedure to the commonly used dynamic analyses. The considered case study consists in a non-seismically designed reinforced concrete frame building under a set of ten real ground motions from Los Angeles.

Keywords: Failure probability, RC buildings, Seismic safety assessment, pushover analyses, dynamic analyses

1. INTRODUCTION

The response of structures to earthquake actions has been the main concern of several investigations in the earthquake engineering domain from the recent years. These approaches can differ according to the structural and demand modeling issues involved in the analyses leading to different levels of accurateness and complexity.

In this context, seismic structural vulnerability assessment methodologies appear as important tools for describing the seismic safety of structures (Baker, J. W., 2007; Cornell, C., et al., 2002; Liel, A. B., et al., 2009). Herein, the contribution of several experimental works concerning seismic behavior of both single elements and the global structures is indeed fundamental to identify the determinant variables of the structural problem. Thus, the seismic structural safety problem is accepted to be fully characterized from the description of the capacity of structures and the seismic demand, especially through the damage distribution over the resistant elements. Notwithstanding reducing the seismic structural assessment to the simple definition of the capacity and demand, there are several aspects to take into account of which the most important is the level of uncertainty involved within its characterization.

So, it is noticeable the importance of a study that focus on the variability of both capacity and demand, and the effect of this non-deterministic behavior for the seismic structural assessment. This can be decisive for the final judgment on the vulnerability of the structure in analysis.

The uncertainty that exists along the seismic safety definition is assessed in this work through the capacity and the demand sides. The variability related to capacity is foreseen to be dependent on the material properties values, while from demand it is considered the uncertainty in the ground motion selection and intensity values. A stochastic approach is followed to account for the randomness feature of these variables, using the Latin Hypercube (McKay et al, 1979) sampling method to generate a set of random samples according to a specific statistic distribution. Thus, it is unquestionable that assuming the capacity and demand as deterministic variables, in seismic vulnerability estimations, can be way far from truth.

Ferry Borges and Castanheta (1972) introduced in their work the structural safety reliability concept, where it was first mentioned the notion of structural risk. This concept of risk consists in the idea that a structure, through its period of life, may assume unfavorable damage states, ranging from specific failure probability values.

The failure probability is an index that reflects the capacity of the structure to respond under a seismic action. This index can be derived globally when it is referred to the structure itself or locally when it represents the failure of a single structural element.

The present methodologies that assess the vulnerability of structures through the computation of a failure probability make use of nonlinear dynamic analyses to determine the seismic response of structures. Nevertheless the advantages of using nonlinear time-history analyses in the sense of an enhanced representation of the real conditions and structural behavior, difficulties arise from its application and inherent theoretical formulation that can lead in some cases to an unfeasible use of these analyses under a probabilistic seismic safety assessment scenario. Such difficulties are mostly presented in the seismic action definition process, namely in the selection of the ground motion records for analysis representative of the seismic activity where the structure is located. However this is not a single limiting aspect, equally important constraints can be found when setting the dynamic parameters as well as in the time consumption of each analysis. These issues restrict spreading the application of probabilistic seismic safety methodologies to the regular works on seismic design and assessment of structures.

Thus, the first goal of this study is to set a probabilistic methodology for assessing the seismic behavior of structures (in particular of buildings), which proposal should, simultaneously, be simple and guarantee the accurate structural vulnerability assessment.

Therefore, the herein proposed methodology derives from the classical structural reliability formulation, being the probability of failure computed from the convolution of the capacity (C) and the seismic demand (D) variables.

Nonlinear static pushover analyses are used in this work to determine if they can be valuable procedures to estimate the seismic response of structures. The use of pushover analyses in a probabilistic seismic assessment methodology is definitely an innovative aspect that ought to be evaluated since it can accurately traduce the real structural behavior, reducing substantially the complexity involved within the entire procedure.

Nonlinear static pushover procedures have been over the past few years studied and proposed by the seismic scientific community, developing efforts towards the establishment of an alternative method in order to overcome the previously mentioned disadvantages of the nonlinear dynamic analyses (Antoniou, S. & Pinho, R., 2004a, 2004b; Barros, R. C., 2010). Herein, the behavior of structures can be analyzed from the capacity or the performance point of view. In the former, it can be highlighted the conventional or adaptive pushover, whether considering or not the influence of higher modes over the analysis due to the stiffness degrading process. On the other hand, from the performance perspective, it should be noticed and considered the influence of aspects such as the seismic action definition (in terms of a response spectrum) and the use of reduction factors to scale elastic response spectrum to the correspondent inelastic one, in such a way to account for the energy dissipation mechanism associated to the cyclic nature of earthquake ground motions, such as hysteretic effects, that is unaccounted for when applying the pushover lateral loads.

Hence, it is understandable the importance of the proposed study to validate the use of the aforementioned nonlinear static analyses within a probabilistic seismic assessment method. In order to accomplish this goal, will be established a comparison in terms of failure probability values obtained from both nonlinear pushover and dynamic analyses.

Beyond the influence of the theoretical concepts behind the seismic structural evaluation methods, it will also be admitted the non-deterministic behavior of the capacity and demand variables in a stochastic procedure.

2. PROBABILISTIC SEISMIC SAFETY ASSESSMENT METHODOLOGY

2.1. Introduction

Seismic assessment and design of any new or existent construction is commonly performed through simplified linear methods, neglecting the dynamic effects characteristics of the earthquake action. In fact, not involving in the analyses the hysteric behavior of structures means that it is disregarded the most important mechanism of energy dissipation, leading to a structural response that is often far from reality. This deviation can be even amplified because the response of structural elements is assumed and modeled in these approaches as having a linear elastic behavior, not including the material nonlinearities. Hence, and in line with the aforementioned in the previous section, it is of noticeable importance the definition of a seismic safety assessment methodology able to reproduce the demand and capacity dynamic phenomena and the available ductility of each element, bearing also in mind the simplicity within its application and formulation, but mainly in the accurate reproduction of the response of structures under a seismic action.

The proposed methodology makes use of the theoretical concepts of the semi-probabilistic approaches (JCSS, 1981), in which the vulnerability is introduced as a margin of structural safety (M) that is implicit in the design code criteria, Eqn. 2.1.

$$M = C - D \tag{2.1}$$

In this technique the proper characterization of the statistical distribution of both variables is fundamental to avoid the inclusion of additional epistemic uncertainties, which affects the computation of the failure probability in a misleading fashion and consequently leads to a wrong evaluation of the vulnerability conditions.

2.2. Formulation of the probabilistic methodology

As already mentioned the present probabilistic seismic safety assessment proposal derives from the validation of a certain probabilistic limit state, LS. This limit state is defined as the boundary between capacity (C) and demand (D), as it can be seen through Fig. 2.1.

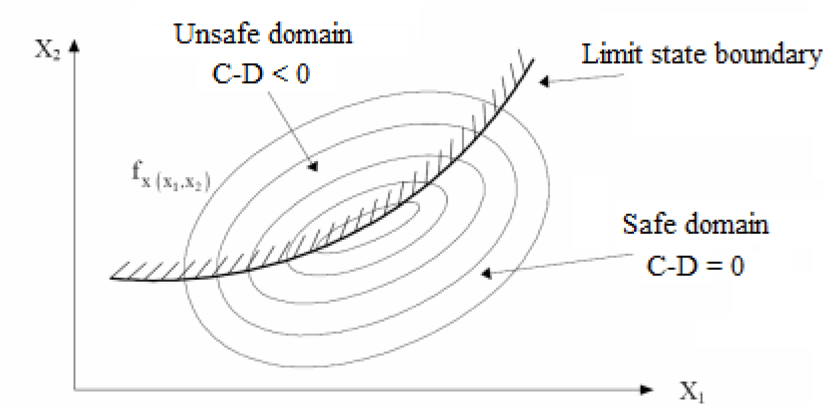

Figure 2.1. Structural reliability problem (Laranja, R.C. and Estevão, J.M.C., 2000)

It is depicted in Fig. 2.1 the three possible stages of the reliability safety problem (Ferry Borges, J., and Castanheta, M., 1972): the unsafe (C-D<0) and safe (C-D>0) domains, and the limit state boundary (C-D=0). These regions determine the vulnerability condition of the structure and are dependent on several random variables (X_i) which in turn influence the capacity and demand.

Therefore, the probability of reaching this limit state, P_{LS}, corresponds to the region of points where the seismic demand surpasses the capacity which means a negative margin of safety, and is commonly known as a structural failure probability, P_f, Eqns. 2.2.

$$P_{LS} = P[C < D] = P_f \tag{2.2}$$

Eqn. 2.2 can be rearranged to

$$P_{LS} = P[C - D < 0] = P_f \tag{2.3}$$

Being C and S random and continuous variables, it is possible to establish a probability density function $f_x(C, D)$.

This function represents each isoprobable elliptical line in Fig. 2.1, being possible to rewrite the failure probability expression of each structural element, Eqn. 2.4.

$$P_f = \int_{g(X)=C-D\leq 0} f_X(x).dx \tag{2.4}$$

X represents a set of random variables x_i, $g(X)$ the limit state function corresponding to a specific collapse mechanism and $f_X(x)$ the probability density function of the vector X. The vector of random variables comprises the entire range of properties that determine the uncertainties involved in the definition of the failure mechanism, the seismic action, the material properties and the geometry of the structure.

Thus, this probabilistic seismic safety assessment methodology is based on the following equation that is obtained from rearranging Eqn. 2.4.

$$P_f = P[g(X) \leq 0] = F_g(0) \tag{2.5}$$

Eqn. 2.5 corresponds to the probability of the function g(X) to rely on the failure domain, in which Fg(0) is the cumulative distribution function of g(X) evaluated at the limit state boundary (X=0).

2.3. Structural capacity

This random variable reflects the strength of a structure and is mainly, but not exclusively, dependent on material mechanical properties and its inherent uncertainty, elements lengths, amount and disposition of the sectional reinforcement, as well as on the material delayed effects and its durability. Indeed, it is clear that the uncertainty of the material mechanical properties, from this set of features, is the one that contributes most to the structural strength variation. Several studies have focused on the range of scatter that is expected to find in steel and concrete parameters (Kappos *et al.*, 1999, Kwon & Elnashai, 2006, Pipa & Carvalho, 1994) using numerical analyses and experimental tests, however under a probabilistic seismic safety assessment model this aspect was not truly included before. These works have pointed out a set of concrete and steel material properties which nature should not be presumed as deterministics, namely the ultimate compressive strength of concrete (f_{cu}), the ultimate compressive strain of concrete (ε_{cu}), the yielding strength of steel (f_{sy}) and the ultimate strain of steel (ε_{sy}).

In this work the structural capacity variable is defined in accordance to the non-deterministic nature of the latter concrete and steel material parameters, assuming the values of coefficient of variation indicated in table 2.1.

Table 2.1. Material properties

	f_{cu}	ε_{cu}	f_{sy}	ε_{sy}
CoV (%)	18.0	35.0	6.0	9.0

A Normal statistical distribution is used to characterize each material property.

A numerical procedure is used to determine the capacity of each structural member, imposing cyclic and increasingly rotations in each end of a structural element (where a plastic hinge is expected to be formed). The structural capacity parameter used in this study is the available rotational ductility at each plastic hinge section.

2.4. Seismic demand

Beyond capacity, demand is the other random variable of the structural reliability problem. In a seismic assessment context this variable reflects the response of structures to an expected seismic action that is probable to occur in the region of interest (which means the site where the structure in analysis is located). In this sense the definition of the seismic action should be consistent and comprise with the seismogenic source zone.

Usually this is represented through a hazard curve that relates the design motion parameter to the probability of exceedance. Hence, this function shows the probability of exceeding ground motions in a time span (return period), for a certain ground motion parameter (for example, PGA). This information is crucial for design and assessment purposes.

Alternatively, the seismic action probability of occurrence can be modeled using a distribution function that returns this probability for a ground motion parameter. In literature this function is ascribed as a Gumbel distribution (the potential applicability is typical extended to flood and meteorological phenomena), settled by a location (mean value) and a scale (β) parameters. Herein in this work, the parameters of the Gumbel distribution were defined to match the expected probability of occurrence for the tectonic region of the Lisbon area.

Therefore, seismic demand is obtained by means of an engineering demand parameter (edp) to express the structural response in terms of deformations, accelerations, or other quantities calculated by simulations of the building to earthquake ground motions. Herein, rotational ductilities will be considered to predict the inelastic seismic behavior at each beam and column extremity (plastic hinge).

2.5. Stochastic procedure for estimating failure probability of structures

Bearing in mind the establishment of a probabilistic seismic vulnerability assessment of structures strategy that takes into account the uncertainties of the material properties and furthermore the seismic action intensity distribution it is introduced a stochastic approach. This approach entails the application of the Latin Hypercube (LH) sampling method underlying the generation of a set of random samples for the capacity and demand variables.

A total of 7 parameters are randomly selected, comprising 4 material properties (f_{cu}, ε_{cu}, f_{sy} and ε_{sy}) and 2 characteristics of the seismic action (accelerogram and peak ground acceleration intensity).

The outcome of the approach is a distribution of performance scores in terms of available and demand ductilities (μ_C). The former are computed using the *n* samples of the material properties through the procedure detailed in section 2.2, for assessing the capacity of each structural section, and depicted in Fig. 2.2.

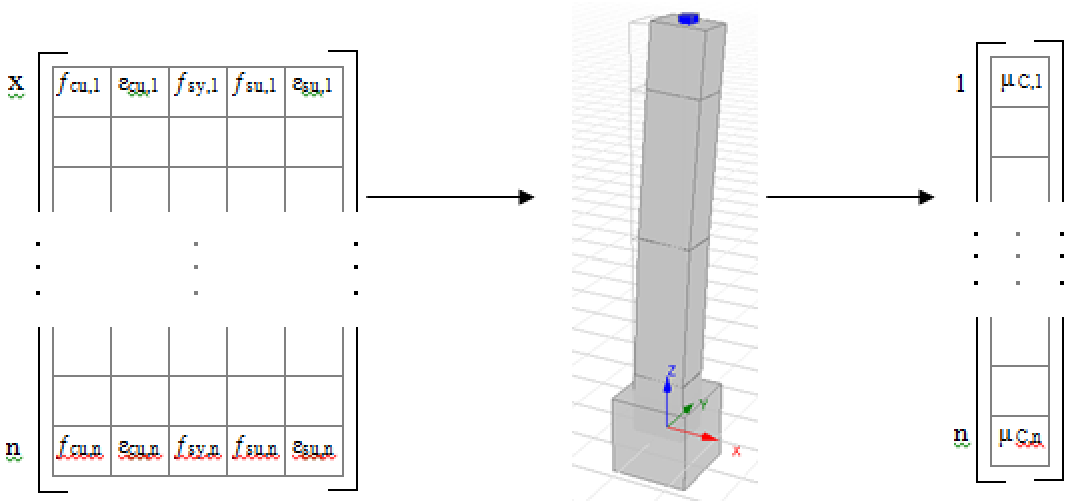

Figure 2.2. Numerical procedure for assessing the available ductility of a single element

The demand ductilities (μ_D), on the other hand, result from the response of the structure itself to a seismic input (performing nonlinear dynamic and pushover analyses), including in the analyses the 7 random parameters. Herein, it is performed a series of *n* pushover and dynamic analyses and the ductilities are assessed in each structural element, Fig. 2.3.

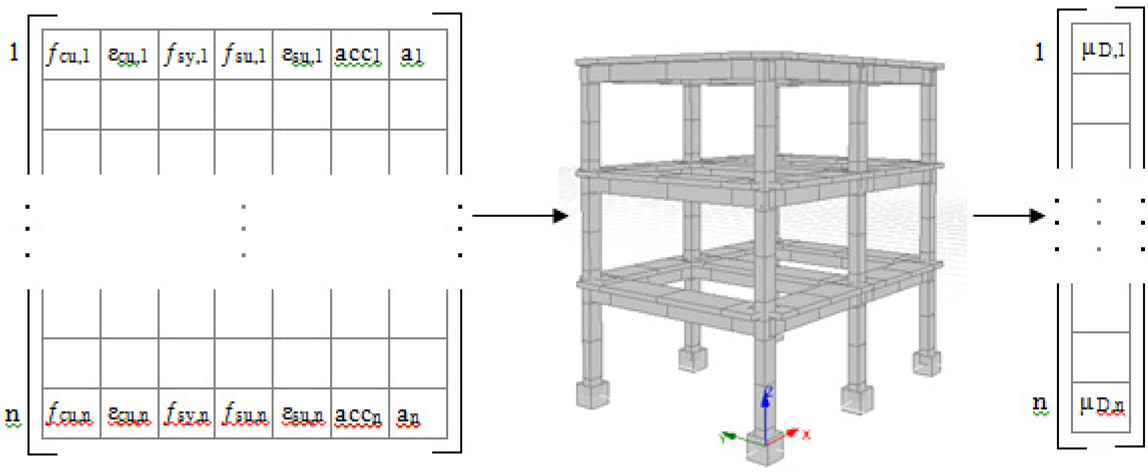

Figure 2.3. Numerical procedure for assessing the demand ductility of each element

The structural vulnerability is expressed as a safety margin and is computed by Eqn. 2.1. A statistical distribution is fitted to the data of safety margin by maximum likelihood, admitting a Normal and a Gumbel function. Hereafter, failure probability is obtained using Eqn. 2.6 of the structural reliability problem, Fig. 2.4.

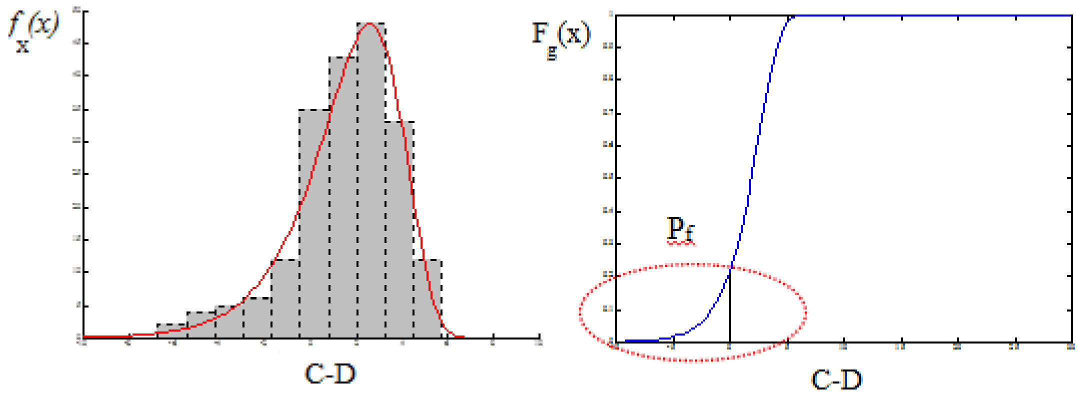

Figure 2.4. Statistical distribution fit to the safety margin data and failure probability definition

The outcome of this approach (the failure probability) is highly dependent on the quality of the distribution fit. Towards this goal two alternative ways were implemented in order to arrive into the best statistical function and with high statistical significance: using 3 robust statistical hypothesis tests (Kolmogorov-Smirnov, Anderson-Darling and Cramér-von-Mises) and a graphical analysis through a probability plot.

This procedure is envisaged to each section responsible for the failure mechanism of the structure that, is in turn, checked through an initial nonlinear pushover analysis. Thus, the failure probability of the structure is measured from the lower bound of the failure probability of a series system.

3. CASE STUDY

3.1. Structural model

The seismic vulnerability of a real reinforced concrete frame building, representative of the European construction, is assessed following the stochastic procedure purposed in this study.

This structure, located in Italy, is a typical construction of the pre-seismic code period, that has been designed taking only in consideration gravity loads. This model, labeled as *mod4*, is an asymmetric three-story RC regular frame structure without masonry infills, with a total height of 9.05 m and with three bays of 4.05 m, 3.0 m and 3.5 m length each, Fig. 3.1.

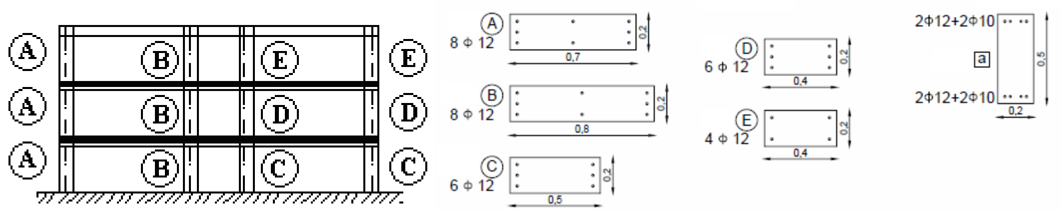

Figure 3.1. Structural model *mod4*

Five column sections were modeled and admitted to be truthful representative of the disposition of the reinforcement bar along the extension of the elements. Furthermore, a single cross section was considered for beams. A lumped mass distribution at each beam-column joint was assumed and included in the modeling of the structure. Additionally, soil-structure interaction effects were not taken into consideration for this study and an elastic damping factor of 2% was considered throughout the structural analyses.

3.2. Ground motion records

The employed set of seismic excitation, used in the nonlinear analyses, is defined by an ensemble of ten records selected from a suite of historical earthquakes scaled to match the 10% probability of exceedance in 50 years (475 years return period) uniform hazard spectrum for Los Angeles (SAC Joint Venture, 1997), which corresponds, in the current endeavor, to the intensity level 1.0. Additional intensity levels, linearly proportional to the latter by a factor of 0.5, 0.75, 1.5, 3.0 and 3.5, were also considered, thus allowing an overview on how results evolve with increasing seismic intensity. The ground motions were obtained from California earthquakes with a magnitude range of 6-7.3 recorded on firm ground at distances of 13-30 km; their significant duration (Bommer and Martinez-Pereira, 1999) ranges from 5 to 25 seconds, whilst the PGA (for intensity 1) varies from 0.23 to 0.99g, which effectively implies a minimum of 0.11g (when intensity level is 0.5) and a maximum of 3.5g (when intensity level is 3.5). The demand spectrum was defined as the median response spectrum of the ten records. Analyses were carried out using the fiber based software SeismoStruct.

4. RESULTS AND CONCLUSIONS

The nonlinear static procedure (NSP) application was carried out according to what is suggested in EC8, which makes use of N2 method (Fajfar, P. and Fischinger, M., 1988). According to that, a first mode proportional invariant load shape has been applied in respect to the computation of the capacity curve of the *mod4* frame, over a conventional pushover technique. Moreover, the accurateness of the computed capacity curves was previously controlled using the results of the structural response obtained from nonlinear dynamic analyses (Marques, M. and Delgado, R., 2010].

The same effort was devoted to evaluate the validity of the application of nonlinear pushover analyses for estimating the failure probability of each structural section. This strategy (comparison of local failure probabilities from dynamic and pushover approaches) was first introduced among the results of capacity and demand ductilities, Fig. 4.1.

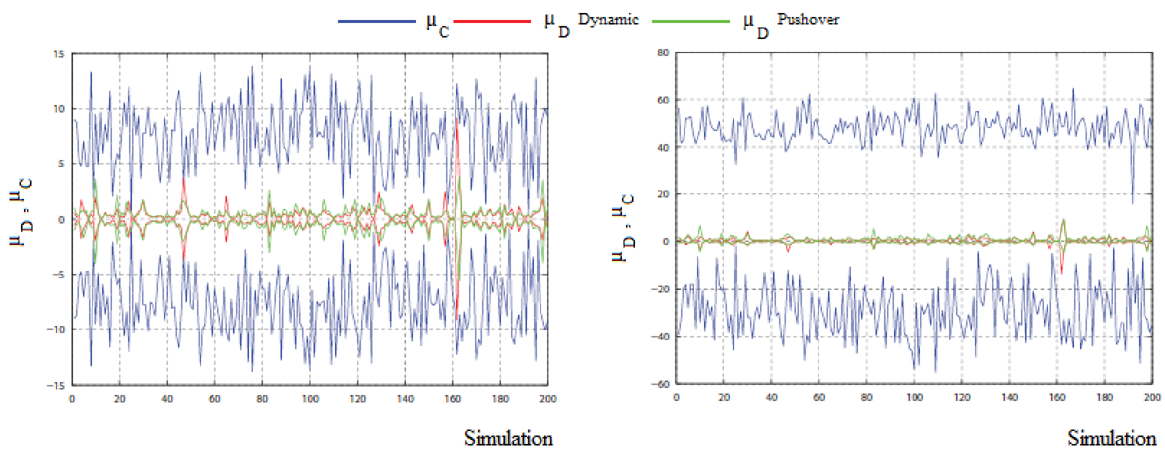

Figure 3.1. Capacity and demand ductilities (plot at left corresponds to a column cross section, whilst the plot at right is for a beam section)

Fig. 3.1 shows a promising performance on the demand ductilities computation using pushover analyses. This first insight on the static nonlinear good shape is found in seismic demand behavior of the two structural elements, for each single sample simulation.

The collapse probability results for the plastic-hinge cross sections of the underlying failure mechanism have stated, according to Fig. 3.1, an almost equal behavior captured from the dynamic and the pushover analyses, which was also already detected on demand ductilities. Differences on the failure probability values obtained from the two adopted nonlinear procedures, when found, exhibit marginally higher probabilities kept from pushover analyses.

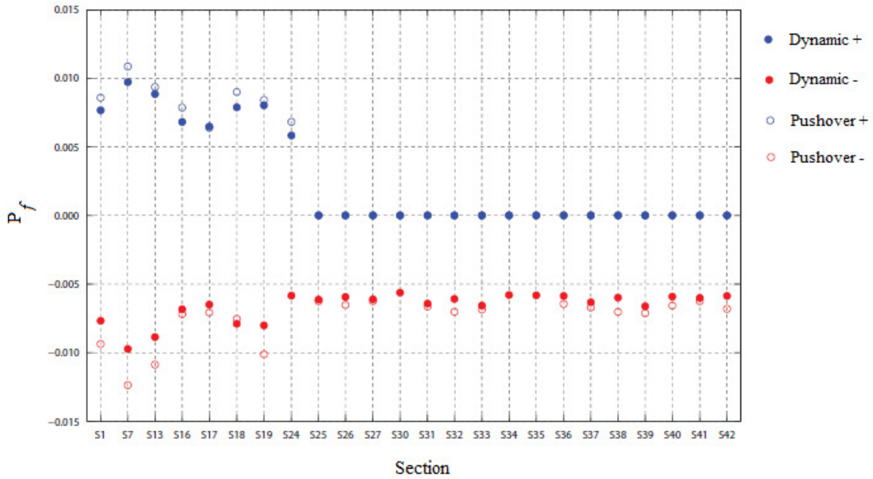

Figure 3.1. Failure probability results for each plastic-hinge cross section

It was also underlined a weaker structural performance to the negative direction of the seismic load, for sections starting on S24.

With the intention to check the vulnerability in a global fashion, the failure probability of the model *mod4* was computed according to the approach referred in section 2.5. Despite being straightforward, this procedure permits to identify the consistency of the assessment of the structure over the two nonlinear methods, as well as to quantify with a single index the structural vulnerability. Therefore, assuming this structure as a series system in which the elements are fully correlated, the structural collapse mechanism is considered to occur when the first plastic hinge is formed. This assumption means that the failure probability of the model *mod4* is the same as the maximum failure probability found within their sections (the most vulnerable cross section). Thus, a global probability of $1x10^{-2}$ is obtained from both pushover and dynamic analyses, maintaining the same association between these two approaches and in line with the results per cross section.

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