COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

STUDY ON THE DEFINITION OF ANALYTICAL FRAGILITY FUNCTIONS FOR CONCRETE BUILDINGS

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Keywords: Fragility, reinforced concrete building, seismic risk, limit states, pushover analysis, incremental dynamic analysis.

Abstract. *Development of earthquake risk mitigation strategies is a major concern of the seismology and seismic engineering communities with the main purpose to evaluate the impact of future earthquakes on society and to be able to define appropriate measures to protect citizens and the built heritage. In this way, seismic risk has been defined as the probability of losses, from human, social or economic, due to earthquake scenarios. The assessment of the vulnerability features of the built environment is a key component of a loss model in order to establish the probability of a given level of damage to a given type of structure, and associated with a specific seismic scenario. Within this evaluation, the use of fragility functions is very important since gather the information of the relationships between building performance and ground motions. This study addresses the analysis of different aspects for the definition of the fragility functions, such as: the intensity measure levels, the type of nonlinear analysis to assess the structural performance, the local and global engineer demand parameters (EDP) to characterize the behaviour of the structure and the damage state thresholds. Herein were considered the recommended provisions for seismic risk and safety assessment of structures, included in Hazus [1], FEMA 356 [2] and Eurocode 8 [3] documents and past research works. A comprehensive comparison on the aforementioned parameters and provisions is followed so as to understand how the accuracy of the derivation of the fragility functions relies on such aspects. Therefore, a three-storey gravity designed reinforced concrete building is considered on behalf of its fragility assessment, including a set of 10 real gound motion records homogenized to match the NEHRP design spectrum, defined with a 10% probability of exceedance in 50 years for Los Angeles, scaled by peak (PGA) and spectral (at the fundamental period of the structure, Sa (Te)) accelerations for 36 intensities. Nonlinear incremental dynamic and nonlinear pushover analyses, with the N2 static procedure, are used and the fragility of the building is defined using three global and local quantities: the maximum interstorey drift, the global drift, and the chord rotation of columns and beams.*

1 INTRODUCTION

The current strategies for seismic risk assessment try to replicate in their models the most complete information existent at the site of analysis, so as to ensure that, in the end, the potential damages induced by an earthquake are accurately estimated.

Thus, in general, we may consider three components of the seismic risk [1]: the hazard, the exposure and the vulnerability. When modelling the seismic hazard one should take into consideration several issues, such as the location, intensity or frequency of the events, with the aim to get the values of probability of exceeding a given level of ground shaking intensity within a given time span [2]. Concerning the exposure, a comprehensive survey on the elements that are potentially subjected to losses in hazard zones (known as assets) is sought [1]. Herein, a good exposure model has to describe the distribution of a set of assets, for example in terms of population or structures, grouped by their similar features (location, taxonomy and value). Finally, in what regards to vulnerability, it is expected that this model depicts in a proper way the characteristics of each asset that make it susceptible to the damaging effects of the hazard. When focusing on the physical side of the vulnerability it is deemed crucial to describe the variation in the distribution of loss with increasing intensity measure levels. Social, economic and political factors should also be catered in a seismic risk assessment [3].

Notwithstanding the large number of studies concerning the aforementioned main individual components of a risk analysis [4-6], due to all the uncertainty sources and difficulties to extend to different geographical, tectonic and construction conditions, there is still a wide effort to be undertaken towards the definition of more truthful risk models.

In line with these observations for seismic risk purposes one may also point out that in what seismic assessment and design activities is concerned it is not difficult to find several issues to enhance its consistency regarding the definition and modelling of the same hazard, exposure and vulnerability components, even within code provisions [7].

For risk and performance analyses it is considerably important to analyse the influence of aspects inherent to the fragility of the exposed assets (mainly structural), such as the definition of criteria for the selection of the seismic input, the considered intensity measure quantity that best correlates to the damage, the levels of acceptance of damages, the engineering demand parameter to characterize the structural behaviour or the thresholds for each change on the state conditions of the structures.

Therefore, this work tries to enforce these concerns following a comparative study on previous proposals. The most up to date and commonly used in Europe and United States seismic code provisions and reports are also assessed in the light of providing a clear overview of its expected outcomes.

2 FRAGILITY FUNCTIONS

2.1 Introduction

The distribution of structural and non-structural losses due to the seismic action effects (usually referred as vulnerability functions) is commonly derived from an initial combination of fragility functions and consequence functions. The latter are generally derived empirically and represent the distribution of loss, given a performance level, while fragility functions are essential tools under seismic loss estimations and describe the probability of exceeding a damage state level for a given structure [8, 9]. Limit states are herein perceived as damage or injury performance levels.

In this respect, one may follow three broad approaches for the definition of fragility functions: empirical, expert opinion and analytical.

Empirical fragility assessment relies on collecting damage data for different building typologies due to past historic earthquake events or following an earthquake, wherein this observation of damage under different levels of ground shaking hazard allows to create motion-damage relationships and extend these findings for structures under similar conditions, from regional to performance ones. The empirical approach is in fact often done for reasons of rapid safety evaluations of structures in the aftermath of an earthquake; for detail damage evaluation of a subset of the total number of buildings; or for detailed post-earthquake surveys undertaken by reconnaissance teams on small samples of the buildings [10, 11], Figure 1.

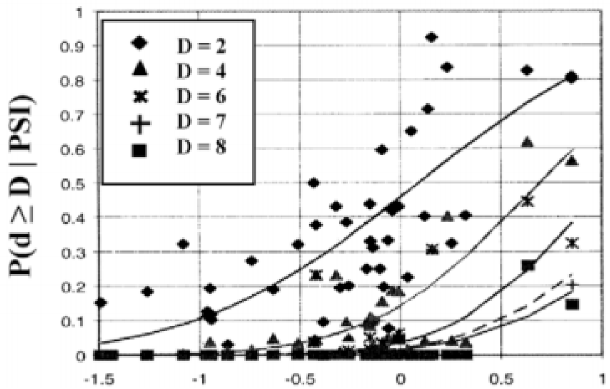


Figure 1: Example of empirical fragility functions for a class of building [12].

Notwithstanding the ease of derivation and the valuable and realistic information gathered from this approach, there are a number of drawbacks [13]. Aspects such as the relatively small number of adequate data points, bias towards damaged buildings or the need to predict the level of ground shaking and its consequent uncertainty, may pretermit the use of empirical fragility functions.

Although less common, the use of expert opinion fragility functions can also be found in literature [14, 15]. The results of this approach are expressed in terms of damage probability matrices and mean damage factor functions. These outcomes describe qualitative the proportion of buildings belonging to each damage grade for different levels of intensity, Table 1.

CLASS A		DAMAGE GRADE					
		0	1	2	3	4	5
INTENSITY DEGREE	V		Few				
	VI		Many	Few			
	VII			Many	Few		
	VIII				Many	Few	
	IX					Many	Few
	X						Many
	XI						Most
	XII						All

Table 1: Example of a damage probability matrix for vulnerability class A [16].

Shortcomings in this methodology are addressed to the vagueness of the matrices, since they are described qualitatively, and to the lack of information for all damage grades for a given level of intensity. To tackle this limitation authors [16] have proposed the use of the Fuzzy Set Theory.

Although the inclusion of real damage data and the experience attained from past seismic events, due to its limitations, the regular use of the aforementioned approaches is arguable when deriving fragility functions for economic and casualty loss estimations. Thus, also a result of the evolution on the computational capacity, the analytical derivation of the fragility of structures is seen as the most interesting, straightforward and thorough alternative. Herein, structures are modelled through numerical models, duly calibrated to include several material, seismic load and geometric phenomena, in order to relay an adequate and realistic behaviour of a structure or class of structures.

2.2 Analytical fragility functions

Analytical methods have often been used to develop building fragility curves, wherein the structural demands and capacities used may be estimated from elastic spectra, nonlinear static and nonlinear dynamic analyses [17]. As it is understandable the accuracy on the definition of the fragility functions is highly dependent on the quality of the structural analysis, so as to reproduce its real behaviour. Consequently, nonlinear material approaches should be undertaken preferably.

2.2.1 Methods for nonlinear analysis

Two main categories of analysis are tacitly seen as the most valuable methods for fragility and performance assessment, due to its exactitude or simplicity, respectively: the nonlinear time-history and the nonlinear pushover analyses.

Nonlinear time-history is the most accurate procedure to capture the behaviour of the structures beyond their elastic capacity [18], which is evident to occur in fragility assessment. Moreover, code provisions governing seismic design and assessment of structures have included for over two decades the use of dynamic analysis [19]. The seismic load is herein introduced by means of a sufficiently large set of ground motion records at the base of the model, in such a way that enables an amply and direct evaluation of the seismic source mechanisms and of the record-to-record variability in structural responses, Figure 2.

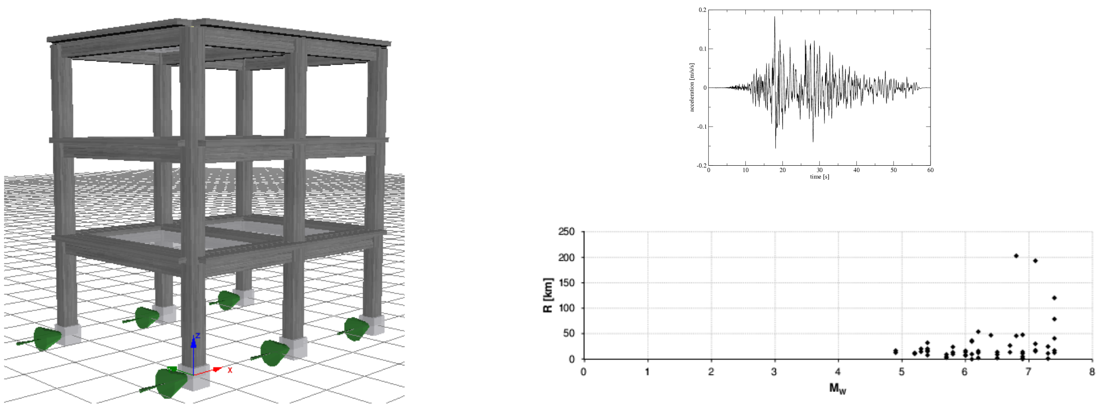


Figure 2: Example of a 3D building modelling for a nonlinear time-history analysis.

Introduced by Vamvatsikos and Cornell [20], the recently emerged Incremental Dynamic Analysis (IDA) is widely used to analyse the structural behaviour under different performance levels to seismic excitations. This method consists in a sequence of nonlinear response history analysis of the structure for a continuously intensity scaled ensemble of ground motions, to cover the entire range, from elastic behaviour to global dynamic instability, of structural response.

However, one of the major drawbacks in both dynamic and incremental dynamic history analyses is the sensitivity of the results obtained to the characteristics of the selected seismic input, in addition to the higher level of complexity, computational demand and the significant required modelling knowledge, besides the appearance of numerically unstable solutions.

Thereby, nonlinear static pushover analysis has appeared as an interesting alternative approach, due to its simplicity and expeditious, guaranteeing also accurate results. Static analysis caters for the global response of the structures by means of a relationship between the total base shear and the displacement of a control node (usually at the top of the building). This capacity curve expresses an envelope of the equivalent outputs of the dynamic analysis by imposing a lateral load pattern in height (at each storey level) to laterally displace the structure stemming the change on its modal properties.

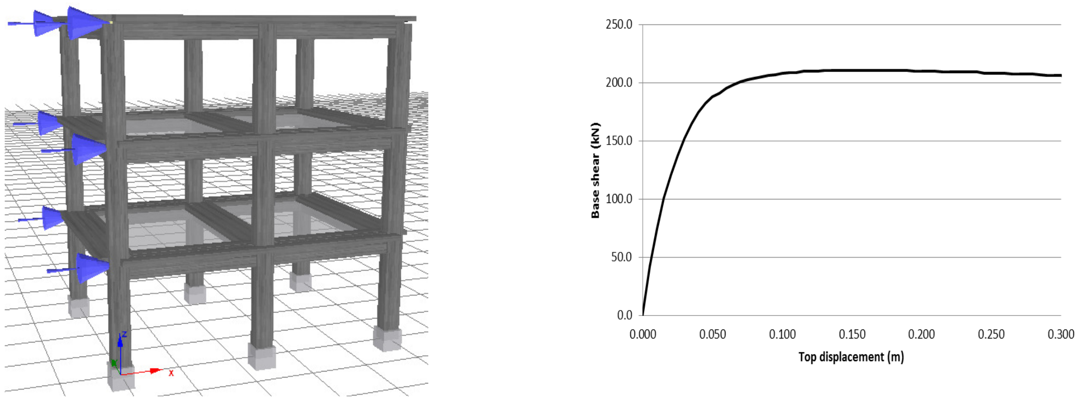


Figure 3: Example of a 3D building modelling for a nonlinear pushover analysis and capacity curve.

Different and enhance approaches haven been proposed concerning assumptions on behalf of the application of the lateral load pattern, thus distinguishing two main proposals: the conventional pushover and the adaptive pushover. The former ones apply a continuously incremented pattern that may follow a uniform, triangular or first mode proportional shape, while on the latter the load shape is changed in each step in accordance to the change on the modal properties of the structure.

Notwithstanding the global structural capacity information given by capacity curves, one may compute the performance of the structure when subjected to a specific intensity level of the seismic input. Therefore, there are a number of nonlinear static procedures (NSP) to estimate the response, among which the main ones are N2 [21], Capacity Spectrum Method [22], Modal Pushover Analysis [23] and Adaptive Capacity Spectrum Method [24]. Herein, the seismic action is introduced by an elastic response spectrum (preferably a code spectrum to vanish potential irregularities on real spectrum). These NSPs diverge on the amount of reducing the elastic response spectrum in order to account for hysteretic degrading effects, on the strategy to reduce the real structure to an equivalent single-degree-of-freedom (SDOF) one and on how *"intersect"* the demand spectra to the capacity behaviour of the structure.

Nonlinear pushover are thus appealing tools, turning into key elements when dealing with massive structural calculations (which is the case when deriving analytical fragility curves), mainly because of their less computational demanding and numerical stability, although its applicability has relevant limitations for irregular structures and do not account for the dynamic effects inherent of the seismic action, moreover one needs to use additional procedures for computing the performance of structures to specific levels of a seismic load.

2.2.2 Damage response measures and limit states

In order to quantify the damage in structures due to the seismic action one may define a specific measure. This is indeed a key stage for the final derivation of the fragility functions and towards this many proposals have been ever since appeared in literature, as well as, in code provisions.

Generally, a division in terms of displacement-based, energy-based and hybrid response quantities is made regarding to the referred damage measures [25]. Likewise, separating these parameters bearing in mind if they represent global or local quantities is also possible.

In light of the most common and reliable damage measures one should point out, for the case of global parameters, the maximum interstorey-drift proposed in FEMA 356 [26] or the global-drift that is used in HAZUS [27]. Herein, interstorey-drift is computed as the relative lateral displacement between floors and is expressed as a percentage of the storey height. On the other hand global-drift appears as the top displacement as a percentage of the building height.

The building performance is evaluated at specific damage levels, also mentioned as limit states, which are function of the referred drifts. In this sense, FEMA 356 proposes three damage performance levels: Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). HAZUS itself suggests four limit states: Slight Damage (SD), Moderate Damage (MD), Extensive Damage (ED) and Complete Damage (CD).

These documents albeit initially including a qualitative description for the aforementioned performance levels, suggest rendering to the class of structure in analysis quantitative threshold values, based on comprehensive review of past works and real damages experienced in structures from past seismic events. For the sake of further application in the present work one presents for a reinforced concrete moment frame class the threshold values of FEMA 356, Table 2, and for a reinforced concrete moment frame, Low-rise, Pre-code design level class the threshold values of HAZUS, Table 3.

Classification	**Structural damage states**		
	Immediate Occupancy	**Life Safety**	**Collapse Prevention**
Reinforced Concrete moment frame	0.010	0.020	0.040

Table 2: Maximum interstorey-drift threshold values suggested in FEMA 356.

Classification	**Structural damage states**			
	Slight	**Moderate**	**Extensive**	**Complete**
Reinforced concrete moment frame, Low-rise, Pre-code design level	0.002	0.005	0.012	0.028

Table 3: Global-drift threshold values suggested in HAZUS.

In what concerns to local quantities, a number of research studies have appeared with proposals to quantify the damage of structural elements when the structures are subjected to earthquake loadings [28, 29]. These works had been catered on seismic codes, endorsing chord-rotations as good estimators of the structural damage in ductile elements. FEMA 356 and Eurocode 8 recall on this parameter, proposing the threshold values for beams and columns presented in Table 4, and the performance Equations 1 and 2 for Slight Damage (LS1) and Near Collapse (LS3) thresholds, while Significant Damage (LS2) limit state is quantified by 75% of the LS3, respectively.

Classification		Structural damage states		
		Immediate Occupancy	Life Safety	Collapse Prevention
Reinforced Concrete moment frame	Beams	0.0015	0.002	0.003
	Columns	0.0050	0.015	0.020

Table 4: Chord-rotation threshold values suggested in FEMA 356.

$$\theta_y = \phi_y \cdot \frac{L_v}{3} + 0.0013 \cdot \left(1 + 1.5 \cdot \frac{h}{L_v}\right) + 0.13 \cdot \phi_y \cdot \frac{d_b \cdot f_y}{6\sqrt{f_c}} \tag{1}$$

Where,

$\phi_y = \dfrac{\varepsilon_y}{h}$	Yield curvature of the end section;
$\varepsilon_y = \dfrac{f_y}{E_s}$	Yield strain;
d_b	Mean diameter of the tension reinforcement;
h	Depth of cross-section;
f_y	Steel yield strength, in MPa;
f_c	Concrete strength, in MPa;
$L_v = \dfrac{M}{V}$	Ratio moment/shear at the end section.

$$\theta_u = \frac{1}{\gamma_{el}} \cdot \left(\theta_y + \left(\phi_u - \phi_y\right) \cdot L_{pl} \cdot \left(1 - \frac{0.5 \cdot L_{pl}}{L_v}\right)\right) \tag{2}$$

Where,

γ_{el}	Is equal to 1.5 for primary seismic elements and to 1.0 for secondary seismic elements;
$\phi_u = \dfrac{\varepsilon_{cu} + \varepsilon_{su}}{h}$	Ultimate curvature at the end section;
ε_{cu}	Ultimate strain of the extreme fiber of the compression zone;
ε_{su}	Ultimate strain steel;
$L_{pl} = 0.1 \cdot L_v + 0.17 \cdot h + 0.24 \cdot \dfrac{d_b \cdot f_y}{\sqrt{f_c}}$	Length of the plastic hinge.

2.2.3 Procedure for the generation of fragility functions

The analytical approach concerning the generation of fragility function is in general established for classes of structures, where the required simulation effort is very considerable, even for a limited number of random variables. However, the aim of the present study is to assess, following a comparative analysis, the fragility of an individual building in regards to the most important steps and parameters and to depict their influence on the derivation of these curves.

Therefore, the key steps on the generation of fragility functions are, as follows:

- the selection of strong ground-motion records consistent with former seismic hazard analyses, in addition to the included intensity demand measure level that best correlates to the conditional probability of damage, usually expressed in terms of peak ground accelerations (PGA) or spectral accelerations at the fundamental period of the structure (Sa(T));
- the definition of the capacity model of the structure, using one of the previously mentioned methods for nonlinear analysis and assessing the response of the structure given shaking hazard;
- the performance damage analysis, estimating through eligible engineering demand parameters the aftermath of the global behaviour of the structure at several seismic intensities;
- the computation of conditional probabilities of the seismic demand (D) placed upon the structure exceeding its capacity (C) for a given level of ground motion intensity (IM) and for a determined performance level;
- the fit of a function to the cumulative conditional probability points referenced to a single limit state (LS). According to Cornell et al [30], a lognormal cumulative distribution function (CDF) is a reasonable assumption for the statistical distribution of the response of the building, as expressed in Equation 3 in terms of an earthquake intensity measure. Hence, one needs to compute along the x intensity measure quantity the median (μ) and the standard-deviation (β) parameters of the CDF, from the Maximum Likelihood Estimation method.

$$P_i\left(LS_i \mid IM\right) = F_i\left(x\right) = \Phi\left[\frac{1}{\beta}\cdot\ln\left(\frac{x}{\mu}\right)\right] \qquad (3)$$

3 CASE STUDY

3.1 Objectives

The methodology presented above will be applied for the fragility assessment of a reinforced concrete building, illustrating further details on key aspects of the approach through a comparative study addressed to:

- the use of PGA and Sa(T) scaling intensity measures;
- the application of nonlinear pushover or time-history nonlinear methods;
- the global and local engineering demand parameter to quantify the structural demand;
- the performance damage limit states proposed in FEMA 356, HAZUS and Eurocode 8 [31].

3.2 Building model

The seismic fragility assessment of a real reinforced concrete frame building, representative of the European construction, is performed following the procedure already presented in the above sections.

This structure, located in Italy, is a typical construction of the pre-seismic code period, which has been designed for resisting vertically-acting gravity loads. This model, herein called *mod4*, is an asymmetric three-storey RC regular frame structure without masonry infill walls, 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, illustrated in Figure 4.

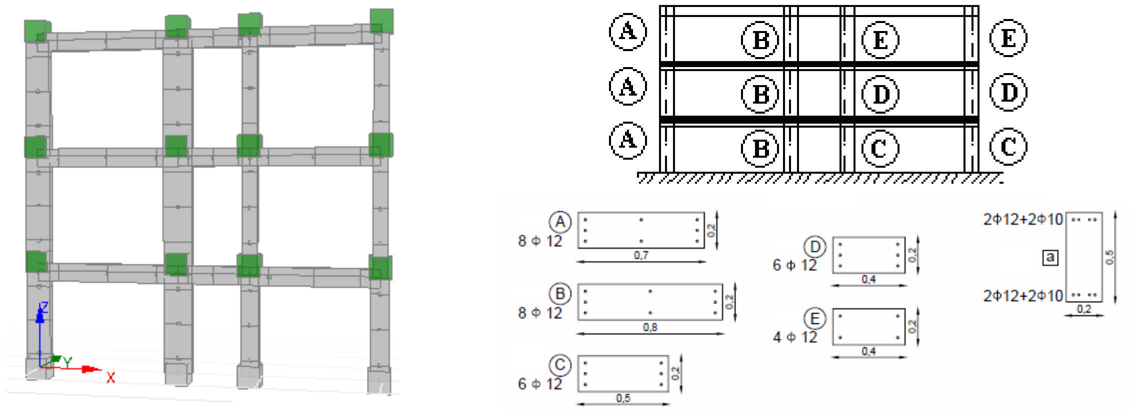


Figure 4: Structural model of mod4 frame building.

Five column sections were modelled so as to truthfully represent the disposition of the longitudinal reinforcement bars along the elements length. Whilst, a single cross section was considered for beams. A lumped mass distribution at each beam-column joint was assumed in the structural model. Moreover, soil-structure interaction effects were not taken into consideration for this study and 2% tangent stiffness proportional viscous damping was considered in the dynamic analyses.

Structural analyses were carried out using SeismoStruct [32], a finite element software capable of predicting the large displacements behaviour of space frames under static or dynamic loading, taking into account geometric nonlinearities and material inelasticity. The nonlinear material behaviour of the structure was captured using a fibre-based model where the inelasticity is considered distributed along the elements length. Modal analysis have concluded that *mod4* is mainly governed by its first mode of vibration, with a fundamental period of T=0.34s and an effective modal mass of 78% of the total mass of the structure.

3.3 Strong ground motion records

The seismic input for deriving fragility functions consists of 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 [33].

The ground motions were obtained from California earthquakes with a magnitude (Mw) range of 6-7.3 recorded on firm ground at distances (D) 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), Table 5.

Label	Record	Mw	D (Km)	Duration (s)	Significant Duration (s)	PGA (cm/s^2)	Sa(T) (cm/s^2)
LA02	El Centro, 1940	6.9	10.0	53.48	24.52	663	1309
LA04	Imperial Valley, 1979	6.5	4.1	39.38	7.09	479	1510
LA06	Imperial Valley, 1979	6.5	1.2	39.09	11.22	230	622
LA08	Landers, 1992	7.3	36.0	79.98	22.24	417	1103
LA10	Landers, 1992	7.3	25.0	79.98	20.72	353	877
LA12	Loma Pietra, 1989	7.0	12.0	39.98	6.40	951	1791
LA14	Northridge, 1994	6.7	6.7	59.98	5.52	644	1824
LA16	Northridge, 1994	6.7	7.5	14.945	7.04	569	1331
LA18	Northridge, 1994	6.7	6.4	59.98	5.30	801	2711
LA20	North Palm Springs, 1986	6.0	6.7	59.98	6.78	968	2311

Table 5: Employed set of records.

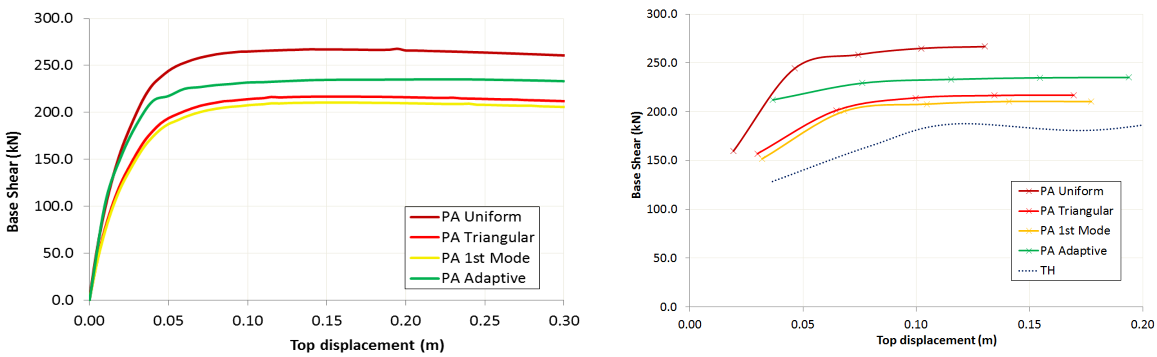
3.4 Background on performance assessment

In the current endeavour two stages were considered regarding the scaling of the ground motion records.

The former admits for the set of ground-motion records and respective spectra five intensity levels, linearly proportional by a factor of 0.25, 0.5, 0.75, 1.0 and 1.25, performing both nonlinear pushover and dynamic approaches, in a total of 50 analyses. This preliminary study enables the comparison between PGA and Sa(T) without introducing additional bias into the study due to individually scaling each record to a target intensity level.

For the second study records are scaled in terms of PGA to 36 intensities levels, considering a maximum acceleration of 2000 cm/s^2.

An initially analysis concerning the performance of pushover analyses (PA) has revealed that no major differences are depicted from adaptive and conventional triangular and first mode proportional load patterns, Figure 5. Likewise, their differences to the median structure responses captured by dynamic time-histories (TH) are identical, and justified to the use of the first version of the N2 method, that disregards the influence of the energy dissipation to the hysteretic damping in the reduction of the elastic spectra.



A) Capacity curves B) Performance assessment

Figure 5: Structure response.

From this point, further pushover analyses are performed admitting an adaptive load pattern.

3.5 Fragility assessment – intensity measures analysis

The fragility curves produced in this preliminary analysis try to call attention to the importance of the intensity measure used for the derivation of these functions. Furthermore the absence of a target intensity level to match each record is also assessed, using solely the original ten ground-motions linearly scaled by five proportional factors.

Hence, these 50 records are sorted according to their PGA and Sa(T) intensity values, and then grouped in sets of 10 elements in terms of ascending intensities. Each new set is considered for deriving the fragility functions, assuming an intensity level equal to the median of the group of records, Table 6.

Set 1		Set 2		Set 3		Set 4		Set 5	
PGA	Sa(T)	PGA	Sa(T)	PGA	Sa(T)	PGA	Sa(T)	PGA	Sa(T)
131	330	241	659	409	990	583	1368	890	2260

Table 6: Set of records and its median PGA and Sa(T) intensities, in cm/s^2.

The performance levels proposed in HAZUS and already presented in section 2.2.2, which consider the global drift measure the engineering demand parameter, are used to develop the fragility functions of the frame. A statistical treatment on the exceedance of each limit state is carried out, as illustrated for the case of pushover analysis in Figure 6, and thereby derived the fragility of the structure.

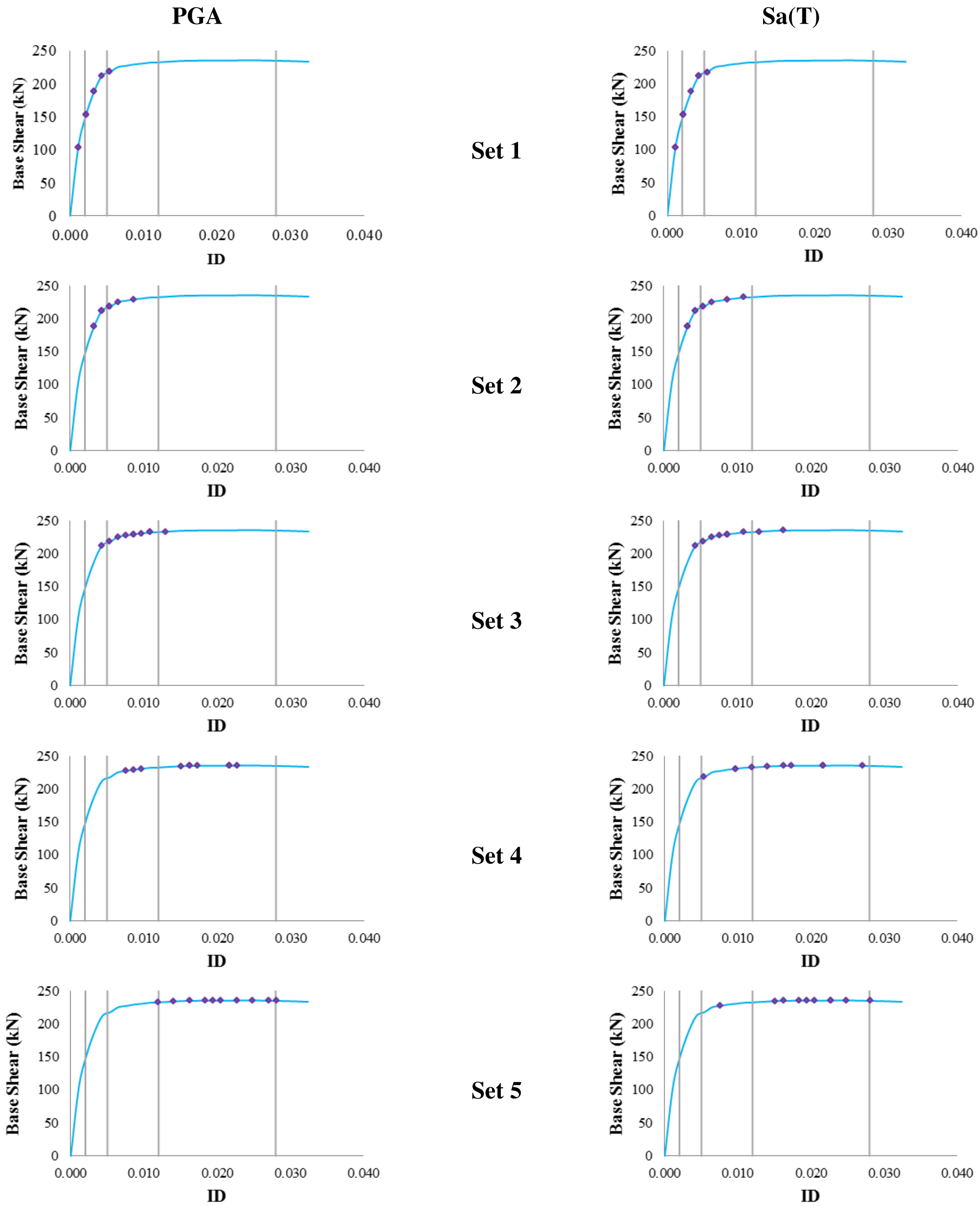


Figure 6: Pushover analysis performance points.

Figure 6 represents the performance of the structure using pushover analysis in regards to the global-drift and total base shear relationship, for each of the 10 ground-motion records contained to each ensemble. From this comparison it is attained the quite close behaviour of the building disclosed for both PGA and Sa(T) measures.

A meaningful comparison regarding the influence of the nonlinear method of analysis is needed for assessing the fragility of the structure with this approach.

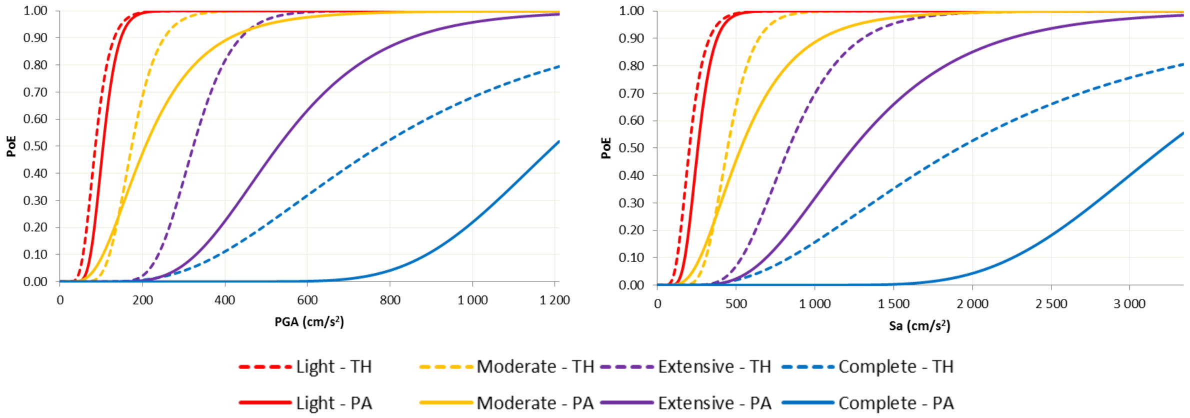


Figure 7: Fragility functions for HAZUS thresholds.

Comparing the fragility curves generated in terms of PGA or Sa(T) no major differences are identified for all the limit states of damage, Figure 7, either in the case of pushover analysis or in the dynamic analysis. Theses outcomes are explained with the similar performance responses of the structure already pointed out (Figure 6), which is a promising conclusion since one of the concerns in seismic risk evaluations is indeed the type of scaling of the seismic input. Although, this statement has to be validated through an extensive analysis, considering a wide set of ground-motion records, intensities and structures.

The conclusions drawn for the intensity measures are not extended to the analysis of the fragility functions derived from the two nonlinear methods – pushover and time-history. Notwithstanding being depicted in Figure 7 a very close fragility behaviour of the frame building for the Light and Moderate limits, it also shows that for high demanding levels both techniques diverge, with a lower fragility being obtained from pushover analyses. This is justified because the nonlinear static procedure used in this assessment, herein the N2 method, has limitations in considering the effects of the hysteretic energy dissipation, likewise the impossibility of the pushover methods to account for the cyclic behaviour effects of the seismic action.

3.6 Fragility assessment – limit states analysis

The aim of the study performed in the current section is to appraise the results of the fragility functions generated from global and local performance levels, at the same that it intends to confirm the aftermath of the preliminary study on behalf of the validation of the conclusions yielded from the nonlinear methods of analysis. Henceforth the 10 real ground motion records are scaled to reach 36 target PGA levels, not exceeding 2000 cm/s^2.

The study begins with a comparison on the aforementioned aspects devoted individualy to the HAZUS global-drift thresholds, Figure 8.

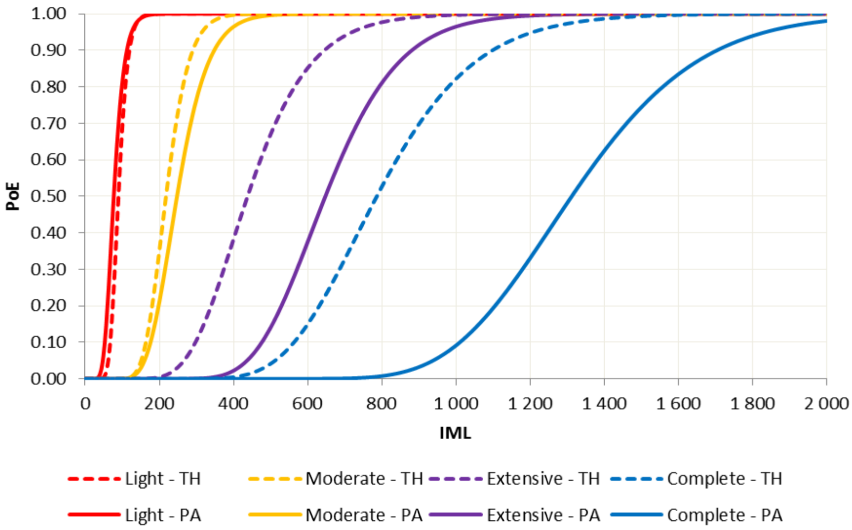


Figure 8: Fragility functions using HAZUS thresholds.

It is also observed that once the structure is responding at more demanding performance levels (extensive and complete limit states) the fragility functions derived from pushover and time-history analysis diverge. This has already been stated in the former section albeit the difficulties to fit a fragility distribution to the five points of probability of exceedance, and is justified by the loss of accuracy of the static analysis for high inelastic demands. Thus no additional comparisons regarding the use of pushover or time-history methods are made since the former seem to be only valid up to global-drift values close to the ones defined at the moderate performance level. Further assessments are only concerned on dynamic analyses of the structure.

The next analysis focus on the derivation of the fragility functions following the limit states proposed by HAZUS, FEMA and Eurocode 8. A straightforward approach is conducted, assembling the results by their structural quantities (drifts and chord-rotations).

The initial analysis on the structural fragility of the model is devoted to the drift-based thresholds of the HAZUS and FEMA 356 documents, illustrated in Figure 9 with continuous and dashed lines, respectively.

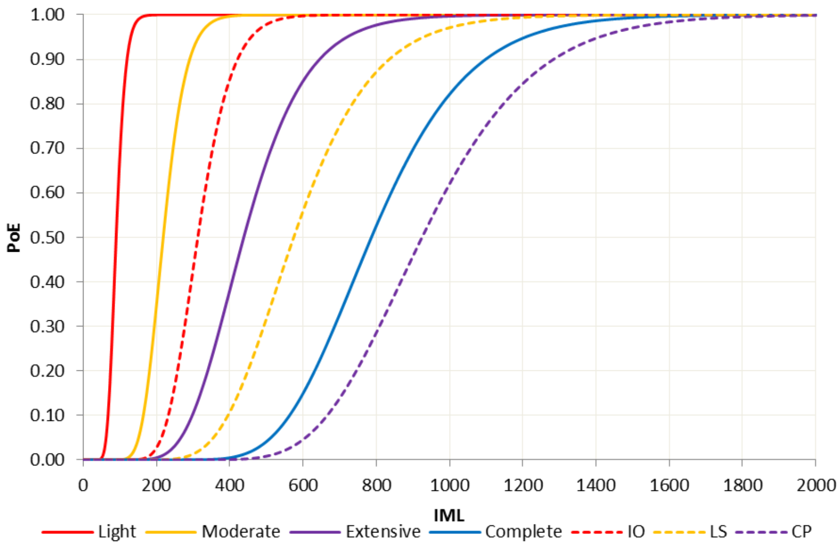


Figure 9: Fragility functions using HAZUS and FEMA drift-based thresholds.

It is noteworthy that fragility functions derived with HAZUS global-drift thresholds are far conservative than the ones obtained for the FEMA 356 maximum interstorey-drift. These differences might be explained with the evolution on the failure mechanism of the structure, which is typical of a soft storey one, and then the expected agreement between maximum inter-storey drift and global drift is not attained. Notwithstanding one may say that the Moderate, Extensive and Complete damage levels of HAZUS are comparable to the respective IO, LS and CP limit states of FEMA.

Moving forward to the analysis of local engineering demand parameters, the impact of the chord rotation thresholds is assessed for the limits proposed in Eurocode 8 and FEMA. This assessment is performed in terms of the first vertical element that reaches the limit values (Figure 10 A) or neglecting the type of structural element (Figure 10 B)

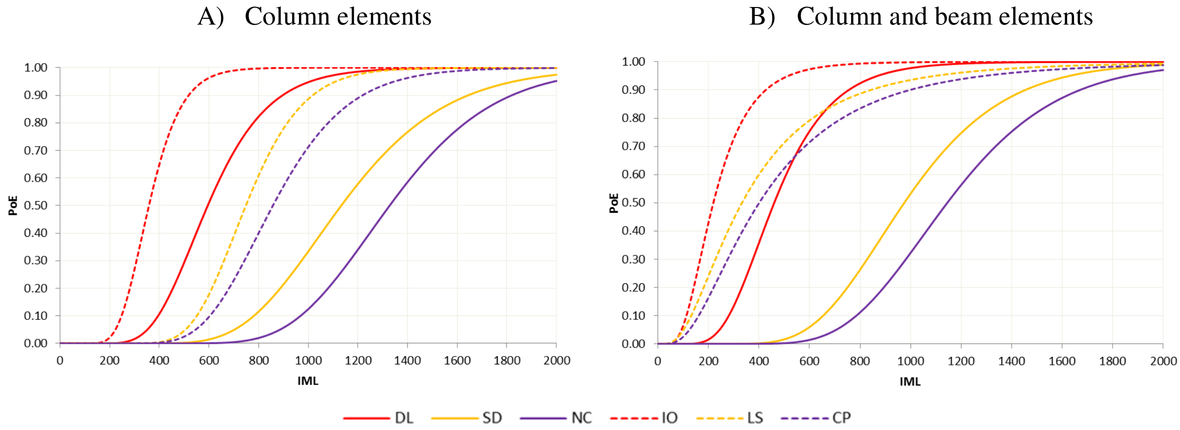


Figure 10: Fragility functions using Eurocode 8 and FEMA chord rotation-based thresholds.

The fragility functions defined in what regards only to column elements are noticeably less demanded, especially for the first limit state, because beams are the first structural elements to yield, even for this pre-coded building. In any case, if FEMA or Eurocode 8 provisions are to be applied one should neglect the type of element. However the most realistic approach bears in mind with the importance of the structural elements for the specific limit states, though if a collapse level is evaluated one should consider for example the first column or entire storey beams that surpass their limit values.

It is also depicted from Figure 10 that FEMA limits lead to more conservative fragility functions. This may be linked to the way how these limit states are established: while FEMA addresses specific values collected from an extensive review in former works, as a function of the structural class but neglecting additional information such as the materials and the dimensions of the elements, Eurocode 8 presents equations 1 and 2 that compute those thresholds at each element according to a set of parameters of the section.

Analysing Figures 9 and 10 it is conspicuously seen that global-based quantities correspond to more restrictive thresholds and high demanded structures. In this sense, HAZUS is the more conservative proposal. Besides considering 4 limit states of damage, the first level of HAZUS has no agreement with the other documents in analysis.

4 CONCLUDING REMARKS

The evaluation and comparison of the methodologies for derivation of fragility functions has been carried out for a pre-coded reinforced concrete frame building. A comparison has been established on the influence of the nonlinear methods of analysis, scaling techniques for the seismic input, intensity measure parameters, engineering demand parameters for the fragility performance levels and thresholds of damage.

The study was separated into two stages concerning: the intensity measures analysis and the limit states analysis.

In general, results have indicated that:

- The fragility functions are not affected by the choice of the intensity measures.
- For more demanding limit states and higher intensity levels the use of pushover analysis, at least with the N2 method for performance assessment, has revealed to be less conservative than dynamic analysis.
- From the above assessed documents, the Eurocode 8 is the one that leads to lower fragility. On the other hand, the use of HAZUS thresholds is reflected in a higher fragility of the structure.

Although several relationships were outlined in the study an extensive number of analyses should be carried, increasing the number of buildings and records. In addition it would be interesting, for the sake of comparison to the observed trens, considering empirical fragility functions for the assessed class of buildings.

ACKNOWLEDGMENTS

Financial support to the first, third and fourth authors is provided by the Portuguese Science and Technology Foundation (FCT – Fundação para a Ciência e a Tecnologia), under the QREN - POPH programme.

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