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ADVANCES IN THE DEFINITION OF FRAGILITY MODELS FOR THE DEVELOPMENT OF RISK-TARGETED HAZARD MAPS

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Abstract

Current seismic design codes require buildings to be designed for a given ground motion intensity level that is determined from a prescribed return period. The implicit assumption is that this design criterion ensures that the probability of collapse of buildings, although unknown, is at least uniform. This is unfortunately not the case. Uncertainties in the fragility of the structure and on the shape of the local hazard curve often lead to an associated seismic risk level that is not only site-specific but also structure-specific, thus invalidating the previous hypothesis.

A new approach for the definition of the design ground shaking, commonly called risk-targeted hazard mapping, aims at estimating the ground motion intensity that leads to a uniform distribution of risk or collapse probability. An essential aspect of this methodology is that for the collapse probability at the design ground motion intensity must be known. This parameter, and the associated dispersion, can be constrained by analysing sets of structures designed according to the same criteria. In this study, a large number of structures designed according to the most up-to-date seismic regulation in Europe has been analyzed. The structures were designed for increasing levels of ground motion intensity (PGA ranging from 0.05g to 0.4g). Tridimentional finite element models have been created and nonlinear dynamic analyses have been performed in order to assess the buildings' seismic performance. Several statistical analyses were performed to estimate boundaries for the collapse probability at the design ground motion, as well as its dispersion. A comprehensive study on the influence of these parameters on the final risk metrics (e.g. annual average loss) was also included. Finally, using the derived fragility parameters we present a suggestion for risk-targeted hazard mapping in Europe.

Keywords: Risk-targeted hazard maps, Analytical fragility, Reinforced concrete moment-frame buildings

1. Introduction

Current seismic design codes require buildings to be designed for a given ground motion intensity level that is determined from a prescribed return period. The implicit assumption is that this design criterion ensures that the probability of collapse of buildings, although unknown, is at least uniform. This is unfortunately not the case. Uncertainties in the fragility of the structure and on the shape of the local hazard curve often lead to an associated seismic risk level that is not only site-specific but also structure-specific, thus invalidating the previous hypothesis. In addition, the seismic events that stoke Christchurch in 2010 and 2011 lead to the 500 and 2500 year return period ground motion in less than six months. This observation raises questions regarding the reliability of current methods for determining the design ground motion. From this discussion it seems obvious that current methodologies for seismic design need revision.

A methodology known as risk-targeted hazard assessment which aims at determining the ground motion intensity that actually leads to a uniform distribution of the seismic risk within a region has been proposed[1, 2]. The target seismic risk level is directly correlated to the risk that a given community or society is willing to accept from a social-economical perspective, and it should be established by decision makers (e.g. politicians and sociologists). Adopting a design methodology based on an acceptable risk level not only ensures a uniform distribution of risk but also overcomes the reliability issues on determining the expected return period for a given ground motion level.

From the aforementioned, it is clear that following a design methodology based on the principles of risk-target assessment has several advantages in comparison with the current procedures. However, a significant obstacle in the implementation of this methodology still needs to be overcome, the definition of fragility curves based on the design ground motion (a_{des}). Low values for the probability of collapse at the design ground motion ($PC|a_{des}$) are to be expected for newly designed structures. However, a literature review has revealed extremely high variability for this parameter with values ranging from 10^{-7} to 10^{-2} [3-7]. These studies considered different types of buildings, designed according to different standards and codes that require the use of values of a_{des} corresponding to very different return periods. Appropriate boundaries for $PC|a_{des}$ and its associated dispersion can be defined by analysing large suits of structures designed according to the same criteria. The collapse probability is usually modelled as a lognormal distribution, which is fully defined by a logarithmic standard deviation, β, and by any quantile of the distribution (e.g. the 50^{th} quantile). High dispersion has also been found for β with proposed values ranging from 0.5 to 1.0. This parameter has a direct influence on the resulting risk-targeted hazard results. To understand this impact, it is fundamental to further investigate the structural fragility of new structures for a wide spectrum of design ground motions.

Moreover, despite the obvious need for collapse prevention when designing and constructing new structures, it is also important to minimize the potential losses due to extensive damage for more frequent events. Observation of past events has revealed regions where modern seismic design regulations are well established, but still high economic losses have been reported. For example, the 1994 Northridge earthquake is deemed as one of the costliest seismic events in recent history, and most of the economic losses came from severely damaged structures, and not due to its limited number of collapses. This aspect has already been featured in some design regulations, such as the Eurocode 8 [8], which establishes a damage limitation for design ground motion corresponding to a probability of exceedance of 50% in 50 years. However, such an approach, once again, leads to an uneven distribution of risk across a given region.

This study investigates the structural fragility of new buildings designed according to the European regulation, within the context of risk-targeted hazard mapping. This goal is achieved through numerical modelling of a number of structures designed considering different seismic hazard levels, which are then utilized to perform several nonlinear dynamic analyses (NDA). The building responses resulting from the NDAs are combined with a damage model to derive fragility functions for two damage states: yielding (that represents the onset of damage) and structural collapse. A comparison is also made between existing fragility functions and those developed herein. Conclusions are drawn regarding the impact that fragility curves with different characteristics have on the annual probability of collapse or of reaching other damage states of interest.

2. Numerical models and ground motion selection

For this study, a pre and post-processing Matlab® [9] algorithm has been developed and all the structural analyses have been performed with the open-source finite element software OpenSees [10]. For the sake of simplicity, the case study buildings are all reinforced concrete moment resisting frames designed according to the most up-to-date European regulations, [8, 11-13]. All the structures are regular in height and symmetric along both horizontal axes. It is acknowledged other lateral load resisting systems could have been considered (e.g. dual systems), however including them would probably widen too much the scope of this study. It should be also noted that for the maximum number of floors considered herein the moment resisting frame is still a relatively common solution for most seismic design.

The concrete class chosen for the structural design has a characteristic strength of 25MPa whilst the characteristic yield stress of the rebar steel considered herein was 500MPa. A permanent load of 6.25 kNm^{-2} has been considered on all floors to reproduce the weight of a reinforced concrete slab of average thickness. Following the guidelines of Eurocode 1-1 [11] for residential buildings, an additional live load of 2.80 kNm^{-2} has also been taken into account in the design stage. For the top floor (roof) the absolute value of the live load has been lowered to 0.40 kNm^{-2}. In addition to the vertical loads, all structures have been designed to withstand the horizontal loading due to the wind excitation, considering a wind velocity of 25ms^{-1} and a Class II terrain, according to the Eurocode 1-4 [12].

To avoid excessive deformations under static loading, all beams have been designed with a minimum height equal to 1/12 of the span length, while the minimum cross section considered for columns was 0.25x0.25 m^2. Standard values for the reinforcing bars have been used (e.g. 6, 8, 10, 12, 20, 25 and 32 mm). The final solution for the rebar pattern was chosen by minimizing the difference between the required rebar area and the actual area while ensuring sufficient spacing between the steel bars. When designing the structural elements, if the cross section of any structural components had to be updated, the minimum increment in the section's dimensions considered was 0.05 m.

Five sets of 10 artificially generated 5-storeys structures designed for increasing levels of ground motion have been analyzed herein. In order to introduce variability in the design, the span length and storey height have been randomly sampled from the probability distributions proposed by Silva *et al* [14]. The structures have been designed for increasing levels of ground acceleration ranging from peak ground acceleration (PGA) of 0.05g (i.e. low seismic hazard) up to 0.40g (i.e. moderate to high seismic hazard). Eurocode 8 [8] performance requirements and recommendations have been followed during the design phase. In order to assess the structural performance of each building, 3D nonlinear finite element (FE) models have been created using the open-source software OpenSees [10]. The models were defined using force-based fibre elements, each with five Gauss-Lobatto integration points. Fig 1 displays the natural uncracked periods of vibration for the different sets of structures computed from the nonlinear 3D FE models.

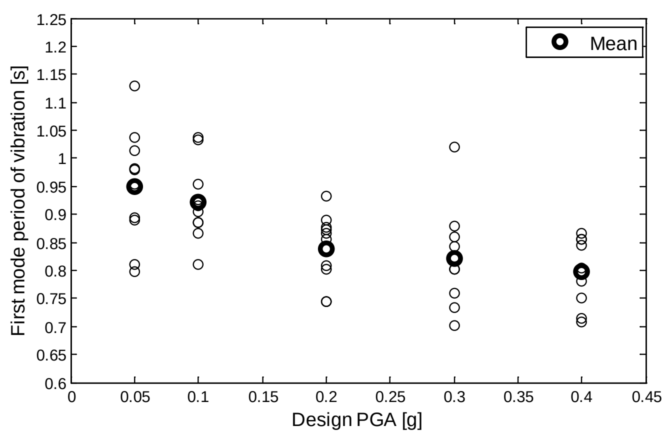

Fig 1 - First mode elastic period for case study structures.

Ground motion is known to be one of the main sources of uncertainty in structural vulnerability assessment [15]. For this reason, special consideration has been given to the selection of the ground motion records. The framework developed by Sousa *et al*. [16] has been used herein to select a large number of accelerograms. This framework is strongly based on the Conditional Spectrum (CS) method developed by Baker and co-workers [17-19]. This method relies on the empirically verified assumption that the set of (log) spectral accelerations (S_a) at various periods follow a random multivariate normal distribution. By defining the target spectral acceleration at target period of vibration one can obtain from disaggregation the parameters, such as magnitude and source-to-site distance, of the controlling scenarios. The values of such parameters inserted in a ground motion prediction model (GMPM) provide the conditional mean and variance of S_a at the vibration periods of interest. The methodology proposed by Sousa *et al*. [16] improves on the original CS method by using more than one ground motion prediction model to estimate the parameters from the disaggregation.

3. Performance and fragility assessment

For this study, a modified version of the Incremental Dynamic Analysis (IDA) [20] often called adaptive IDA or multiple stripe analysis [21] has been applied. The chosen intensity measure for ground motion selection in this study was the spectral acceleration at the mean period of each set of structures. The seismic loads were introduced using scaled real ground motion records, applied to the structure's foundations in both horizontal directions. The combination of effects given by the bidirectional loading was done with one of the horizontal components being multiplied by 0.30, whilst the other remained unchanged, as recommended by the Eurocode 8 [8].

The damage states thresholds used for fragility assessment have been calculated from the individual capacity curve of each structure computed from adaptive pushover algorithms [22]. Two limit states were considered in this study, yielding (marking the onset of damage) and collapse, which are the damage states of significance for risk-targeted hazard assessment. Yielding was assumed to have occurred for the interstorey drift level at which the relationship with the normalized base shear departs considerably from linearity. The exact interstorey drift threshold was computed from bilinearization of the capacity curve. Similarly to previous studies (e.g. [6, 14]), herein the buildings were considered to have reached their ultimate lateral load bearing capacity when a 20% reduction in base shear was observed. In addition the analysis of the capacity curves (see Fig 2) has revealed excessive ductility in some structures, with this drop in base shear only occurring for maximum interstorey drift levels at which the structures are most certainly unstable and/or unrepairable. For these structures the drift thresholds proposed by Ghobarah [23] for ductile moment resisting frames have been followed. Considering both of these criteria for collapse assessing warrants that not only the development of a failure mechanism is captured but also includes, in an indirect way, considerations on the economic viability of restoring heavily damaged structures to their initial capacity.

Table 1 presents the different drifts threshold used herein to compute the structure's fragility curves sorted by damage state and design ground motion. Most of the structures reached the yield damage state for an interstorey drift level of around 1%, which explains the similar fragility curves depicted in Fig 3 for the yield damage state. As expected an increase in the collapse drift threshold is observed as the design ground motion level is incremented.

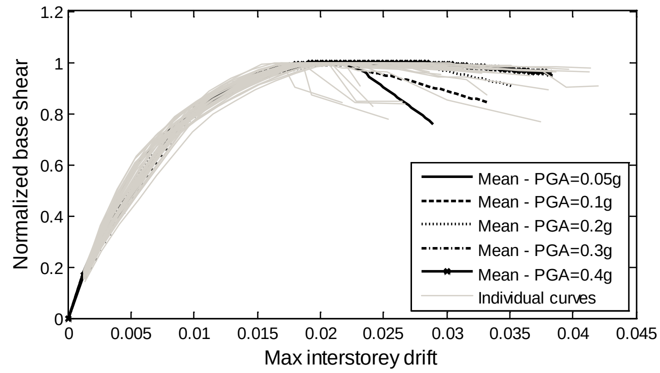

Fig 2 - Capacity curves.

Table 1 - Interstorey drift thresholds

Str.#	a_{des}=0.05g		a_{des}=0.10g		a_{des}=0.20g		a_{des}=0.30g		a_{des}=0.40g	
	Yield	**Coll.**	**Yield**	**Coll.**	**Yield**	**Coll.**	**Yield**	**Coll.**	**Yield**	**Coll.**
1	0.014	0.024	0.012	0.032	0.012	0.024	0.012	0.022	0.013	0.033
2	0.012	0.022	0.012	0.019	0.014	0.039	0.013	0.026	0.013	0.030
3	0.010	0.032	0.014	0.024	0.012	0.028	0.012	0.034	0.013	0.020
4	0.005	0.026	0.012	0.031	0.010	0.025	0.012	0.041	0.013	0.031
5	0.012	0.025	0.012	0.030	0.012	0.020	0.012	0.031	0.013	0.035
6	0.010	0.019	0.010	0.032	0.010	0.025	0.012	0.020	0.007	0.037
7	0.006	0.046	0.007	0.024	0.012	0.033	0.012	0.034	0.013	0.041
8	0.012	0.030	0.007	0.026	0.012	0.030	0.013	0.033	0.008	0.038
9	0.012	0.024	0.007	0.024	0.012	0.040	0.012	0.029	0.013	0.030
10	0.012	0.026	0.012	0.033	0.007	0.033	0.008	0.027	0.013	0.035

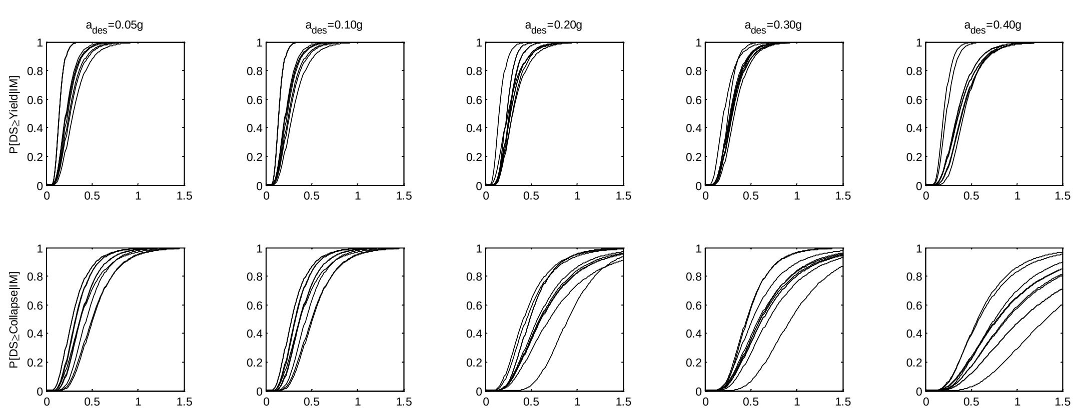

Fig 3 - Fragility curves.

4. Investigation on the probability of collapse at design ground motion and associated dispersion

An alternative manner to represent the fragility curves that provides more useful information within a risk-targeted hazard assessment framework is to use the probability of collapse at the design ground motion, in combination with a measure of dispersion. Through a variable transformation one can compute fragility curves using peak ground acceleration as the intensity measure from the fragility curves in Fig 3. Assuming a lognormal CDF hypothesis for the fragility curve computing the probability of collapse at the design ground motion is simply a matter of estimating the ordinate at a_{des}. From the values listed in Table 2, it has been found that the average value for the probability of collapse at the design ground motion ranged from 10^{-5} for a_{des}=0.05g up to 10^{-3} for a_{des}=0.40g with the most frequent value being under $2x10^{-3}$. Former studies on this subject have suggested different intervals for the probability of collapse at the design ground motion. For example, in a study focused on buildings in France, Ulrich *et al* [6] has proposed an acceptable interval for probability of collapse at the design ground motion between 10^{-7} and 10^{-5}. Other studies, e.g. [2], have suggested intervals with results closer to the ones computed herein, therefore the interval proposed in this study is deemed suitable. It must be noted that lower values (order of magnitude lower than 10^{-8}) have been found for the lowest design ground motion. However, at this design ground motion, the seismic loading was not always the dominant horizontal excitation when comparing to other horizontal loads, namely wind excitation. Some of the most commonly used probability density models have been fitted to the probability of collapse at the design ground motion (see Fig 4-Left). The results indicate that the probability of collapse at the design ground motion follows a beta or gamma distribution. Regarding the dispersion on the fragility curves (evaluated from the lognormal standard deviation) it has been found that the average β parameter ranged from around 0.60 up to 0.80 when using peak ground acceleration (see Table 3). The values found are within the range previously proposed in former studies. Similarly to what has been presented for the probability of collapse at the design ground motion, an investigation on the possibility of modelling β through a probability density function (see Fig 4-Right) suggests that a lognormal model could be adequate.

The results presented in this section provide reliable boundaries for the fragility parameters to be used within a risk-targeted hazard assessment framework. It should be noted that findings presented herein are useful for computing fragility curves for R.C. frames for any region in Europe, provided that the buildings are compliant with the Eurocodes.

Table 2 - Probabilities of collapse at the design ground motion

Str.#	$P[a_c<a_{des}]$				
	a_{des}=0.05g	**a_{des}=0.10g**	**a_{des}=0.20g**	**a_{des}=0.30g**	**a_{des}=0.40g**
1	$2.413x10^{-6}$	$6.312x10^{-4}$	$8.602x10^{-5}$	$5.346x10^{-3}$	$3.075x10^{-3}$
2	$3.645x10^{-6}$	$2.370x10^{-5}$	$8.664x10^{-4}$	$6.120x10^{-3}$	$3.527x10^{-3}$
3	$1.404x10^{-7}$	$1.045x10^{-3}$	$1.150x10^{-4}$	$1.002x10^{-2}$	$1.263x10^{-3}$
4	$7.959x10^{-5}$	$3.721x10^{-3}$	$1.465x10^{-3}$	$1.106x10^{-3}$	$1.082x10^{-3}$
5	$2.417x10^{-7}$	$8.708x10^{-7}$	$1.416x10^{-4}$	$5.227x10^{-3}$	$3.219x10^{-4}$
6	$1.051x10^{-4}$	$2.552x10^{-5}$	$1.465x10^{-3}$	$1.620x10^{-4}$	$4.124x10^{-3}$
7	$1.384x10^{-7}$	$5.226x10^{-3}$	$1.030x10^{-3}$	$1.002x10^{-2}$	$3.252x10^{-4}$
8	$5.105x10^{-10}$	$6.788x10^{-5}$	$8.082x10^{-5}$	$5.081x10^{-4}$	$5.857x10^{-3}$
9	$6.400x10^{-6}$	$5.226x10^{-3}$	$6.964x10^{-4}$	$1.267x10^{-2}$	$1.280x10^{-3}$
10	$7.103x10^{-8}$	$2.969x10^{-5}$	$5.954x10^{-5}$	$7.778x10^{-3}$	$3.219x10^{-4}$

Table 3 - Lognormal standard deviation

Str.#	Lognormal standard deviation (β)				
	a_{des}=0.05g	**a_{des}=0.10g**	**a_{des}=0.20g**	**a_{des}=0.30g**	**a_{des}=0.40g**
1	0.643	0.657	0.601	0.770	0.849
2	0.640	0.590	0.823	0.825	0.813
3	0.639	0.729	0.688	0.917	0.701
4	0.724	0.736	0.787	0.814	0.738
5	0.605	0.523	0.622	0.785	0.730
6	0.715	0.639	0.787	0.534	0.850
7	0.575	0.747	0.819	0.917	0.761
8	0.523	0.614	0.646	0.702	0.938
9	0.702	0.747	0.962	0.978	0.705
10	0.565	0.628	0.628	0.813	0.730

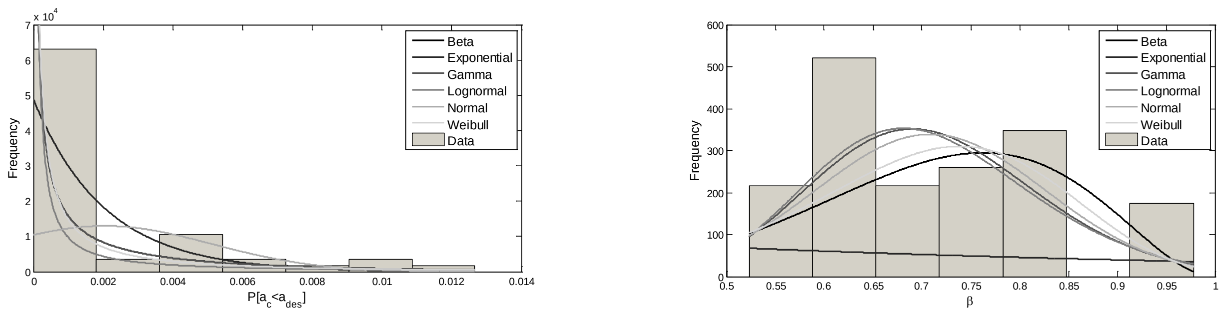

Fig 4 - Histograms and probability density function fitting Left) Probability of collapse at the design ground motion; Right) Lognormal standard deviation

5. Computing mapped ground motion

As previously mentioned, the hypothesis upon which modern design codes were created does not guarantee a uniform distribution of seismic risk within a region. Ideally, one should adjust the design considering a given risk level, and not simply the prescribed ground motion level. This section presents a study on the computation of the ground shaking level that indeed ensures a uniform distribution of risk. Three locations in Europe have been selected based on their prescribed design ground motion (i.e. ground motion with 10% probability of exceedance in 50 years). The design peak ground acceleration for these locations ranges from 0.10g to 0.40g. Fig 5 depicts the hazard curves for the selected locations based on the findings of the European project SHARE (www.share-eu.org).

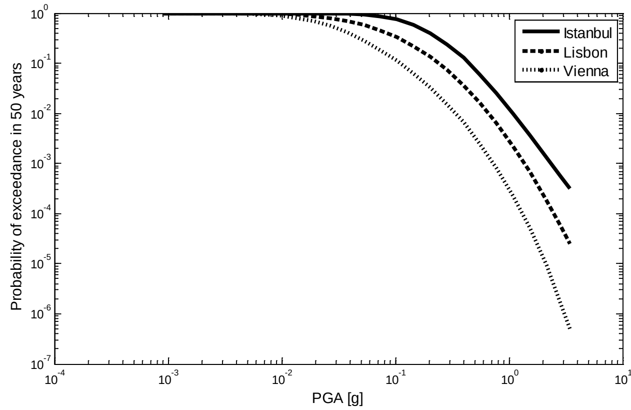

Fig 5 - Hazard curves for the selected locations.

In order to compute the mapped ground motion, the admissible risk threshold has been established as an annual probability of collapse of 2×10^{-5}, which is a value similar to what has been proposed in former studies e.g. [2, 4]. To compute the annual probability of collapse, the iterative method proposed by Eads *et al.* [24] has been used. In this method, the hazard curves are firstly converted from probability of exceedance into annual rate of exceedance versus acceleration. The resulting curve was then divided into segments and the rates associated with the central acceleration value of each segment were calculated. For each building, we then extracted the probability of collapse, PC, at the acceleration corresponding to the central value a_c of each segment. The sum of the product between the rate of occurrence of a_c and the collapse probability PC leads to the annual rate of collapse. The fragility functions required for this computation have been established by sampling one thousand pairs of $PC| a_{des}$ and β from the probability density models plotted in Fig 4. Although Silva *et al* [2] have stated that $PC| a_{des}$ and β are often correlated with, for example, low values of β being usually associated with equally low values *of* $PC| a_{des}$, no correlation between the parameters was considered. Consequently, it is acknowledged that some of the combinations may be unrealistic. Starting from the 475 years return period ground motion level; the annual probability of collapse was computed and compared with the admissible threshold. The mapped ground motion level was adjusted until the relative difference between the computed and acceptable annual probability of collapse was less than a given tolerance (considered to be equal to 5% in this study).

Taking the average value for the mapped ground motion (see Table 4 and Fig 6) it seems that the design ground motion for all three locations needs to be increased for the admissible annual probability of collapse considered herein. Silva *et al* [5] has performed a similar analysis for the entire European territory and has observed that, for example, in the case of Vienna, the design ground motion could in fact be lowered for the same level of acceptable annual probability of collapse. However contrary to what has been executed in this study, Silva *et al* did not consider variability in parameters used to compute the fragility curves. It is recognized that the mapped ground motion is highly sensitive to both $PC| a_{des}$, and β (proven by the variability observed on the histograms plotted in Fig 6) thus different results were to be expected. Furthermore, the maximum value for the probability of collapse at design ground motion assumed herein is two orders of magnitude above the one considered by Silva *et al* (i.e. this study included more fragile structures) and the annual probability of collapse threshold considered herein is slightly lower (albeit in the same order of magnitude). Thus not only different results were to be expected but also the computed mapped ground motion should generally be higher, however the influence of these differences seems to be reduced with the increase on the prescribed ground motion. Relative good agreement with the results from Silva *et al* was found for the city of Lisbon for which both studies suggest the design ground motion to be around 30% higher than the 475 years return period ground shaking level currently enforced by the design codes.

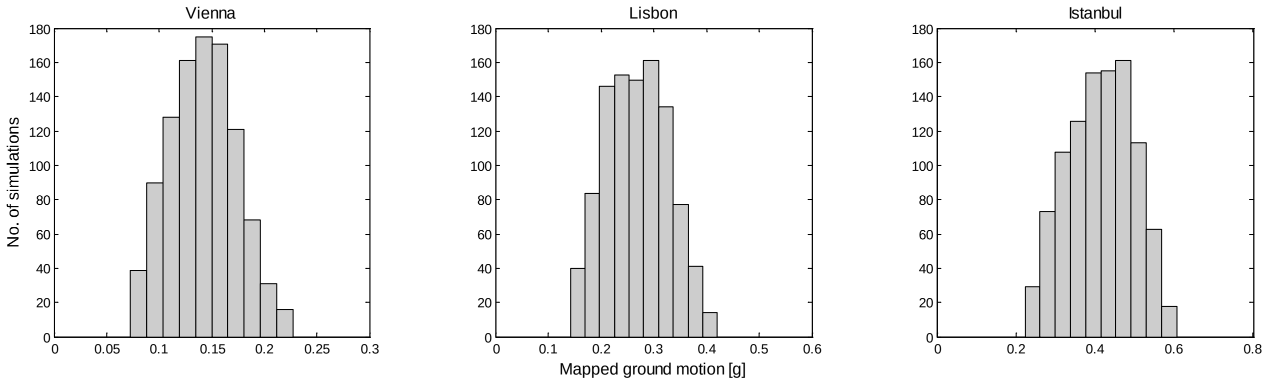

Fig 6 - Histograms for the mapped ground motion

Table 4 - Mapped ground motion results

Location	**Mapped Peak Ground Acceleration [g]**			
	Min.	**Max.**	**Mean**	**CoV.**
Vienna	0.073	0.227	0.142	0.222
Lisbon	0.142	0.421	0.268	0.220
Istanbul	0.223	0.606	0.414	0.201

CoV. - Coefficient of variation

6. Final Remarks

In this study an analysis on the influence of structural fragility on the probability of collapse at the design ground motion of reinforced concrete moment-frame structures designed according to the European regulations was performed. Sets of 5-storeys regular buildings have been analysed through nonlinear dynamic analyses and the respective fragility curves have been computed. The results presented herein allowed estimating suitable parameters for the derivation of fragility curves for structures for any region in Europe, granted that the buildings were designed according to the Eurocodes.

An investigation on the probability of collapse at design ground motion has shown that this parameter, on average, ranges from 10^{-5} for low seismic hazard regions to 10^{-3} for regions with higher seismic hazard. These values are similar to other previously published studies, e.g. [4], but higher than those proposed by Ulrich *et al.* [6]. It must be noted that the buildings designed considering low ground motion led to probability of collapse in the order of 10^{-8}. In these cases, the design was generally conditioned by other horizontal excitation besides earthquake excitation (namely wind loads). These findings suggest that in some locations with very low expected earthquake hazard, the design recommendations enforced by the codes and the lateral load capacity required by other sources of horizontal excitation could be sufficient to ensure a sufficient seismic capacity.

Probability density functions were fitted to for both the probability of collapse at design ground motion ($P[a_c<a_{des}]$) and the lognormal standard deviation (β). These probability models have been used to compute the mapped ground motion for three European cities chosen based on their prescribed design ground motion. The selected sites have a design ground motion (i.e. the ground shaking with 10% probability of exceedance in 50 years) ranging from 0.10 g to 0.40g. A threshold of $2\text{x}10^{-5}$ for the annual probability of collapse was deemed acceptable. The mean mapped ground motion for all the three locations show that in order to achieve the acceptable annual probability of collapse, structures should be designed for a ground motion higher than the 475 year return period ground motion. Comparing the results shown in this study with former studies on this subject, e.g. [5], the mapped ground motion computed herein is generally higher. This result is a direct consequence of having considered more fragile structures (i.e. with a higher probability of collapse at the design ground motion)

and a slightly lower admissible annual probability of collapse than the preceding studies. The dispersion in the mapped ground motion depicted in Fig 6 and the maximum amplitude of values in Table 4 are a measure of the sensibility of the risk-targeted hazard assessment methodology to the fragility parameters since no variability on the hazard curve has been introduced.

Concluding, the results of this study contribute to the understanding of the seismic performance of new structures designed according to modern codes, and consequently to the increase of the robustness and reliability of the risk-targeted hazard methodology.

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