Investigation of Structural Fragility for Risk-targeted Hazard Assessment

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ABSTRACT: This paper analyzes the structural fragility of buildings designed according to the European regulation, in order to identify key aspects that impact risk-targeted hazard assessment. The performance of new buildings is assessed through 3D finite element models and non-linear dynamic analysis. These results were used to compute fragility functions for the damage states of interest for risk-targeted hazard assessment. Conclusions are drawn regarding their influence on the expected annual probability of collapse or of exceeding damage states of interest. The results presented in this study permit a better understanding of the seismic behaviour of new buildings, and consequently of the increase in the reliability brought by the adoption of the risk-targeted hazard methodology.

1. INTRODUCTION

Modern seismic codes require new structures to be designed for a ground motion intensity level, a_{des}, associated with a prescribed return period of exceedance under the assumption that the resultant collapse probability is uniform throughout a given country. The uncertainty on the collapse probability distribution and the variability on the shape of the hazard curves often results on a seismic risk level that is site- and structure-specific, thus invalidating the previous assumption (Silva *et al.* 2015).

A methodology initially proposed by Luco *et al*. (2007) aims at estimating a design ground motion that indeed leads to a uniform risk nationally, and in accordance with a risk threshold that can be established by politicians, sociologists, or other decision makers. Despite the clear advantages in following such an approach, an important obstacle in its implementation still remains. The building ultimate capacity, a_c (expressed in terms of maximum ground motion intensity) of newly designed buildings is, in general, unknown and can be thought as a random variable. A building collapses when the ground motion level, a, exceeds the building capacity. Hence, given than the true value of the capacity is unknown there is a probability, $P[a>a_c]$, that the ground motion a induced by an earthquake at the building site exceeds a_c for whatever value of a_c (including a_{des}, the value used for design). The capacity a_c 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 50th quantile, namely the median, $\hat{a}_c$ for which P[a_c<$\hat{a}_c$]=0.5). This quantile is traditionally chosen as the design ground motion a_{des} and since the safety factors included in codes make a_{des}<$\hat{a}_c$, sometimes significantly lower, then P[a_c<a_{des}]<0.5 often by a very large amount. The probability that the true but unknown capacity a_c is indeed lower than a_{des}, as well as plausible values β for the dispersion of a_c, can be found by analysing large suites of buildings that are designed according to the same code. A review of existing studies demonstrates a range for β between 0.5 and 1.0, and from 10^{-7} to 10^{-2} for the latter (Douglas *et al.* 2013, Luco *et al.* 2007, Silva *et al.* 2014, Ulrich *et al.* 2014). These studies, however, consider different types of buildings, designed according to different codes that require the use of values of a_{des} corresponding to very different return periods. When similar periods are considered this large variability of P[a_c<a_{des}] is drastically reduced, e.g. (Fiorini *et al.* 2014). Regardless, a large variability remains both in suggested values of β and in P[a_c<a_{des}], and this variability has a direct impact in the resulting risk-targeted hazard results. To understand such an impact, it is fundamental to further investigate the structural fragility of new structures for a wide spectrum of design ground motions.

Furthermore, 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 are 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 (CEN 2010b), which establishes a damage limitation requirement for an additional, lower 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 assessment. 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, NDAs. The building responses resulting from the NDAs are combined with a damage model to derive fragility functions for damage states (e.g., yielding that represents the onset of damage), the most important being 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. STRUCTURAL DESIGN

For this study a framework for pre and post processing the results has been developed within a Matlab® (MathWorks 2013) environment and all the structural analyses have been performed with the open-source finite element software OpenSees (McKenna *et al.* 2000).

The structures used herein are regular and perfectly symmetric both in height and in plan and have been designed to be compliant with the most recent European design regulations. Two sets of reinforced concrete structures have been considered, each one with three bays and four frames. The first one is composed by three storeys whilst the second group has five storeys. All structures exhibit three metres of storey height and five metres of span. The concrete class considered in this study has a characteristic compressive strength of 25 MPa while the chosen characteristic yield strength of the rebar steel was 500 MPa.

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(CEN 2009) 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 order 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.

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 (CEN 2010a).

Three different design levels of peak ground acceleration, PGA (a_{des}=0.0g, 0.2g and 0.4g) have been considered herein. The first value of 0.0g means that the structure was designed without taking into account the effects of seismic loading. Of course, even such structures have a capacity to withstand lateral forces, albeit limited. For those structures that have been designed considering seismic loading, Eurocode 8 (CEN 2010b) requires them to meet the terms of not collapsing for a seismic action with a return period of 475 years. Being regular structures, a simpler procedure with 2D elastic frames and lateral forces has been used during the design stage, which is also in agreement with the common practice. In order to compute the design lateral loads, all the structures were assumed to have been constructed on a rock site. Both types of seismic action (Type I and Type II) in Eurocode 8 have been taken into consideration and the one that led to the higher base shear was utilised in the design process. All the structures designed with Eurocode 8 were assumed to have medium ductility.

The columns and beams were designed considering the standard values for the reinforcing bars (i.e. 6, 8, 10, 12, 20, 25 and 32 mm). The final solution for the rebar pattern was chosen to minimize the difference between the required rebar area and the actual area, while ensuring sufficient spacing between the steel bars.

3. NUMERICAL MODELING AND PERFORMANCE ASSESSMENT

In order to assess the seismic performance and the collapse probability for different levels of ground motion, nonlinear 3D finite element models have been developed with the open-source software OpenSees (McKenna *et al.* 2000). The models were developed using force-based fibre elements with five Gauss-Lobatto integration points. Table 1 lists the natural periods of vibration for the two sets of structures.

Table 1. First modal period of vibration [s].

No. of storeys	*a_{des} [g]*		
	0.0	*0.2*	*0.4*
3	*0.76*	*0.55*	*0.46*
5	*1.12*	*0.91*	*0.77*

A modified version of the Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002) has been applied. The seismic load has been introduced using scaled 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 (CEN 2010b).

4. GROUND MOTION SELECTION

Ground motion is known to be one of the main sources of uncertainty in structural vulnerability assessment, e.g. (Shome and Cornell 1999). For this reason, special consideration has been given to the selection of the ground motion records.

The framework developed by Sousa *et al.* (2014) 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 (Baker 2011, Jayaram *et al.* 2011, Lin *et al.* 2013). This method is based on the empirically verified assumption that the set of (log) spectral accelerations at various periods follow a random multivariate normal distribution. By defining the target spectral acceleration at the fundamental period of vibration, $S_a(T_1)$, 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.* (2014) improves on the original CS method by using the full distribution of magnitude and distance given by the disaggregation to compute an 'exact' spectra, rather than the mean or median value of these parameters. Having assumed Lisbon as the location for the structures, the GMPMs of Atkinson and Boore (2006) and of Akkar and Bommer (2010) have been used herein for the required hazard computations.

5. FRAGILITY ASSESSMENT

5.1. Capacity assessment and damage thresholds

To allocate each structure into a damage state after the nonlinear dynamic analyses, a standard criterion based on interstorey drift thresholds has been considered herein. To compute these drift limits, the results of an adaptive pushover analysis have been used. 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

Figure 1 presents the evolution of the base shear (normalized by its peak value) with the maximum interstorey drift. All the structures exhibit a softening behaviour for an interstorey drift of approximately 1%. As expected, the structures designed for the lowest ground motion acceleration level considered herein have the steepest descending branch of the capacity curve, or reach a numerical non-convergence for a lower ultimate drift, which translates into a lower level of ductility.

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 structures were considered to have reached their ultimate load bearing capacity when a 15% drop in the base shear capacity from its maximum value was observed. Figure 1, shows that some structures may possess high levels of ductility and the 15% drop in shear capacity only appears after very large maximum interstorey drift levels at which the structures are certainly unstable and un-repairable. For these structures the generic thresholds proposed by Ghobarah (2004) have been adopted.

Table 2 describes the different drift thresholds used in this study to define the fragility functions presented in section 5.2.

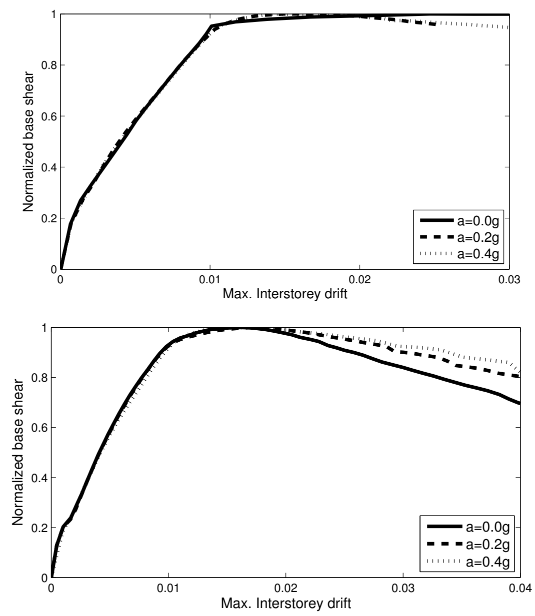

Figure 1. Capacity curves Top: 3 storey structures; Bottom: 5 storey structures.

Table 2. Interstorey drift thresholds [%] for the onset of damage and the collapse limit states

a_{des}	3 storey frame		5 storey frame	
[g]	Yield.	Coll.	Yield.	Coll.
0.0	1.0	2.5	1.0	3.0
0.2	1.0	3.0	1.0	3.0
0.4	1.0	3.0	1.0	4.0

5.2. Fragility curves

Applying the damage criteria discussed in the preceding section, several fragility functions (i.e. the probability of exceeding a given damage state conditional on ground shaking intensity) were derived as depicted in Figure 2 and Figure 3. The two figures show the same set of fragility curves but using firstly $S_a(T_1)$ and secondly PGA as ground motion intensity measure (IM).

Unsurprisingly, the structures explicitly designed to withstand some level of ground shaking exhibit better performance than the structures that were designed for a lower ground motion.

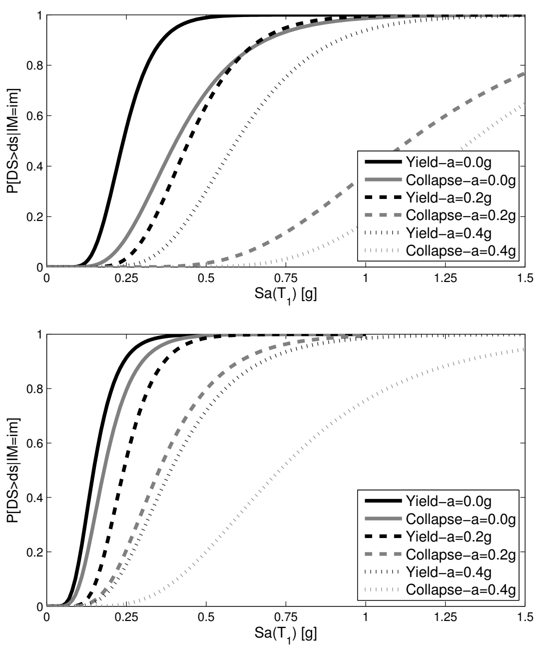

Figure 2. Case study fragility curves Top: 3 storey structures; Bottom: 5 storey structures. (IM=Sa(T_1)).

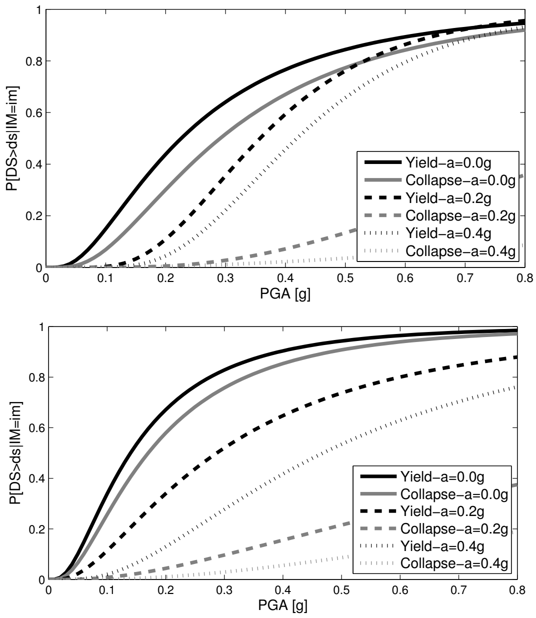

Figure 3. Case study fragility curves Top: 3 storey structures; Bottom: 5 storey structures. (IM=PGA).

The analyses of the lognormal standard deviation of a_c from all the fragility curves in Figure 2 revealed rather small values of β ranging from approximately 0.35 up to 0.45. On the other hand, the β values of the fragility curves computed using PGA as the ground motion IM that are plotted in Figure 3 were always higher (up to approximately 0.8) than those computed using $S_a(T_1)$ as the representing IM. This behaviour is due to the fact that PGA is a less efficient IM than $S_a(T_1)$ in predicting maximum interstorey drift, and the selection method used in this study has been optimized for spectral acceleration. Some authors, e.g. (Douglas *et al.* 2013), have considered a value of β=0.5 as an appropriate lower boundary for the variability in the fragility curves for a building typology, while others have proposed values in the order of 0.8, e.g. (Luco *et al.* 2007), as an appropriate upper limit for this parameter. While the curves computed with PGA all had the β values within this interval, the ones obtained when considering $S_a(T_1)$ were lower than the minimum suggested by the literature. This is most likely due to the small and rather homogeneous sample of structures considered in this study.

For all the fragility functions, it was possible to observe that, for the same design ground acceleration level, the three storey structures demonstrated a higher median value of the capacity for both damage states. For example, for the $S_a(T_1)$ case (Figure 2) the three storey building designed for a=0.4g showed a median capacity of 0.60g for yielding and 1.33g for collapse compared to 0.39g and 0.73g for the five storey building designed for the same ground motion intensity.

5.3. Probability of collapse at the design ground motion

In the introduction we discussed that there is a probability, usually very small, that the true but unknown capacity a_c is indeed equal to or lower than a_{des}. If that were the case, the occurrence of an acceleration equal to a_{des} at the building site would bring the structure to collapse. After the calculation of the PGA fragility functions, we can compute such collapse probabilities at the design ground motion level a_{des}. Such values are presented in Table 3.

Table 3. Probability of collapse at a_{des}.

No. of storeys	*a_{des} [g]*	
	0.2	*0.4*
3	*$5.20x10^{-3}$*	*$2.21x10^{-2}$*
5	*$3.95x10^{-2}$*	*$5.57x10^{-2}$*

Analysing the values given in Table 3 one can conclude that the values are in the range of 10^{-3} to 10^{-2}. These values are in the same order of magnitude of others found in the literature, e.g. (Silva *et al.* 2015), but are significantly higher than those suggested by Ulrich *et al.* (2014). For similar structures (3storey-3bay-4frame designed according to Eurocode 8) Ulrich *et al* have suggested values for the probability of collapse at the design ground motion ranging from 10^{-7} to 10^{-5}. After an analysis of the existing bibliography, e.g. (ASCE 2005), it appears that values of the collapse probability at a_{des} given by Ulrich *et al.* are in the same order of magnitude of those required to some critical facilities, such as nuclear power plants, and, therefore, probably too conservative when applied to standard structures.

It is clear that the value of the probability of collapse at the design ground motion must be properly estimated before any risk targeted analysis is carried out in order to ensure realistic results.

6. COMPUTING ANNUAL PROBABILITY OF EXCEEDING A GIVEN DAMAGE STATE

6.1. Numerical method

In order to compute the design ground motion that may lead to a target acceptable risk, we used the iterative method proposed by Eads *et al.* (2013) that is summarized in the paragraphs below.

As mentioned earlier, we assumed that these buildings are located in Lisbon on a rock site. The hazard curves (see Figure 4) that are required to compute the annual probability of collapse were extracted from the results of the European project SHARE (www.share-eu.org). These hazard curves from the two GMPMs were first converted from probability of exceedence into annual rate of exceedance versus acceleration. Then they were used to compute a single mean annual rate of exceedance curve through a weighted average operation where the weights were those assigned by SHARE to the two GMPMs. 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, P[a>a_c], at the acceleration corresponding to the central value a_c of each segment. The product of the rate of occurrence of the acceleration a_c times P[a>a_c] provides the annual rate of collapse conditional on the occurrence of an acceleration equal to a_c at the site. The annual probability of collapse is simply the sum of these products for all possible values of a_c.

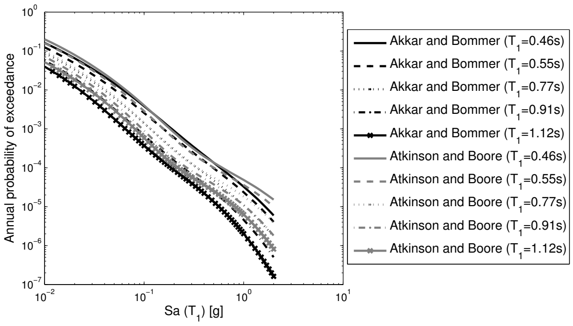

Figure 4. Hazard curves used for this study ($IM=Sa(T_1)$).

6.2. Results and discussion

The annual probabilities of exceeding onset of damage and of collapse computed for all buildings using the methodology just presented are listed in Table 4 and Table 5, respectively.

Table 4. Annual probability of onset of damage ($IM=Sa(T_1)$).

No. of storeys	a_{des} *[g]*		
	0.0	*0.2*	*0.4*
3	*2.50x10^{-4}*	*1.34x10^{-4}*	*1.04x10^{-4}*
5	*2.70x10^{-4}*	*1.38x10^{-4}*	*1.09x10^{-4}*

Table 5. Annual probability of collapse ($IM=Sa(T_1)$).

No. of storeys	a_{des} *[g]*		
	0.0	*0.2*	*0.4*
3	*9.50x10^{-5}*	*1.67x10^{-5}*	*1.07x10^{-5}*
5	*1.78x10^{-4}*	*7.34x10^{-5}*	*2.97x10^{-5}*

Predictably, for a structure of a given total number of floors the annual probability of reaching one of the two limit states decreases with the considered design ground motion, a_{des}. Also it appears that structures of different heights designed to the same a_{des} seem to have similar values of the annual probability of reaching or exceeding the same limit state, here yielding or collapse. This is probably due to the fact that both sets of structures experience yielding at almost the same interstorey drift threshold and exhibit similar post-yield behaviour.

7. FINAL REMARKS

This paper studies the structural fragility of buildings designed according to the European regulation, in order to identify key aspects that impact the development of risk-targeted hazard maps, such as the uncertainty in building capacity and the annual probability of reaching or exceeding relevant damage states.

Two sets of regular reinforced concrete frame structures of three and five storeys designed according to the most recent regulations in force in Europe have been considered in this study. Within a given group of structures the differences in element dimensions and number of rebars were caused exclusively from the design ground acceleration level considered. Numerical finite-element models have been constructed and subjected to a nonlinear dynamic analysis in order to assess the building's seismic performance.

The results presented herein show that the probability of collapse at the design ground motion, a_{des}, for buildings designed according to the newest regulation should be around 10^{-3} to 10^{-2}. These values are similar to those other previously published studies, e.g. (Luco *et al.* 2007), but significantly higher than those proposed by Ulrich *et al.* (2014). The latter study for similar buildings has suggested values ranging from 10^{-7} to 10^{-5}. Hence it is recommended that future investigations on risk targeted design ground motions execute thorough sensitivity analysis on buildings with different characteristics in order to correctly constrain the value of this parameter.

To conclude, 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. However, more research is needed.

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