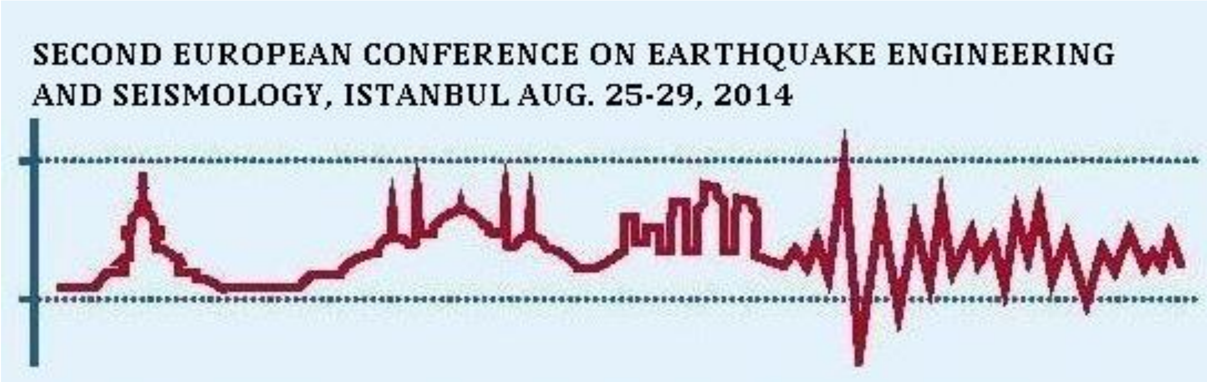

DEVELOPMENT OF MULTI IM BASED FRAGILITY FUNCTIONS FOR EARTHQUAKE LOSS ESTIMATION

Luis SOUSA[1], Vitor SILVA[2], Mário MARQUES[3], Helen CROWLEY[4] and Rui PINHO[5]

ABSTRACT

Fragility functions are a fundamental component in the process of seismic loss estimation, describing the probability of exceeding a number of damage states as a function of a ground motion parameter (IM). In this context, recent efforts have been made in the development of alternative IMs, with the aim of including a wider range of ground motion characterizing information in the definition of intensity. However, the consideration of multiple IMs in the analytical derivation of fragility curves has been subject of limited investigation. This study presents an innovative approach for the development of analytical fragility curves, which relies on a weighted function of a number of intensity measures, along the spectrum of building response. Thousands of nonlinear dynamic analyses were performed in a 2D environment, in which hundreds of reinforced concrete frames are simulated using a Monte Carlo approach, according to geometrical and material variability of typical pre-code reinforced concrete buildings of Portugal. Seismic action variability is foreseen through the selection and scaling of appropriate sets of natural records, following the most recent proposals for linking nonlinear dynamic analysis back to probabilistic seismic hazard assessment (PSHA). As opposite to the generally accepted lognormal scalar-based representation of fragility, a proposal for multiple IM based functions is presented. The prediction of structural damage throughout the considered seismic intensity range is thus affected by a weighted function of a set of intensity parameters that characterize the structure-specific dependence on different ground motion characteristics.

INTRODUCTION

One of the many challenges in using analytical models to predict structural response for fragility assessment is the choice of seismic input to use in numerical simulations. Distinct key characteristics of ground motion such as spectral shape (e.g. Baker and Cornell, 2006) and peak ground motion (e.g. Bradley *et al*., 2009a) have been demonstrated to influence the prediction of demand and capacity of nonlinear systems in a significant manner; with the associated uncertainty being potentially overshadowed by the variability in seismic excitation (e.g. Shome and Cornell, 1999).

Bradley *et al.* (2009a) illustrated that none of the commonly used intensity measures (IMs) are *sufficient* with respect to the distribution of ground motion characteristics – namely, magnitude (M), distance (R), and *epsilon* (ε) - expected at a given site, as determined by probabilistic seismic hazard analysis (PSHA). In the context of hazard compatible ground motion characterization, the dependence of nonlinear analysis results on the suit of selected records appears to be clear, as demonstrated by

[1] PhD Candidate, Faculty of Engineering, University of Porto, Porto, lmcsousa@fe.up.pt
[2] Dr., Department of Civil Engineering, University of Aveiro, Aveiro, vitor.s@ua.pt
[3] Dr., Faculty of Engineering, University of Porto, Porto, mariom@fe.up.pt
[4] Dr., European Centre for Training and Research in Earthquake Engineering, Pavia, helen.crowley@eucentre.it
[5] Dr., European Centre for Training and Research in Earthquake Engineering, Pavia, rui.pinho@eucentre.it

Haselton *et al.* (2007) when evaluating the influence of *epsilon* in the collapse fragility of a large number of structures.

This study presents the proposal of an innovative framework for the analytical derivation of fragility functions, whereby several IMs representing measures of spectral intensity, peak ground motion and spectral shape are conjugated in order to account for the effect of different ground motion characteristics along the spectrum of building response. The General Conditional Intensity Measure (GCIM) approach proposed by Bradley (2010a) is employed in the selection and scaling of records for increasing levels of intensity primarily defined in terms of spectral ordinates at the structure's fundamental period of vibration; according to which the conditional distribution of a set of IMs considered to be pertinent to the assessed structures' response is determined. The spectrum of all rupture scenarios contributing to seismic hazard at the interested site – Lisbon, Portugal – is accounted by means of the relative contribution established by seismic hazard disaggregation (Bazzurro and Cornell, 1999) of magnitude, distance and ground motion prediction models, as formulated by Lin *et al*. (2012). The hazard calculations were performed using OpenQuake, the open-source software for seismic hazard and risk analysis (Silva *et al*., 2013; Pagani *et al*., 2014) developed by the Global Earthquake Model (GEM) initiative.

Thousands of nonlinear dynamic analyses are performed within a probabilistic framework developed by Silva *et al.* (2014a), where hundreds of reinforced concrete distributed plasticity frame models are simulated in a 2D environment; using a Monte Carlo approach according which to the variability in geometric and material properties of typical two, five and eight-story pre-code reinforced concrete buildings in mainland Portugal is taken into account. Structural response is assessed in terms of peak global drift, establishing the scope whereby seismic intensity and cumulative percentage of buildings in each level of structural damage are related, by means of a weigthed function of the most efficient set of IMs.

NUMERICAL MODELS

Reinforced concrete construction accounts for approximately 50% of the total building stock, hosting 60% of the population in Portugal. Following the work developed by Silva *et al.* (2014a), in which material and geometric properties of the most representative Portuguese reinforced concrete moment resisting frame building typologies were characterized, the numerical models considered herein consist of two, five and eight story typical buildings constructed before 1958 (pre-code), year in which the first seismic design provisions were enforced. Dynamic properties are characterized by a mean fundamental period of vibration of 0.26, 0.45 and 0.70 seconds, respectively, as a result from random generation of assets with respect to geometric and material statistical distributions (Silva *et al.*, 2014a) – figure 1. The amount of reinforcement is calculated following the pre-code regulations and practices for ultimate and serviceability limit states; for each asset, in accordance with the sampled material and geometric properties.

To keep the computational effort in a reasonable level, each asset is represented by a single infilled frame with three bays, taking into account the assessed building's lateral load resisting system. Each frame was modeled in 2D environment, with force-based distributed plasticity elements formulation. For the sake of synthesis, numerical considerations adopted with regard to: elements cross section discretization and integration points; material constitutive relationships; p-delta effects; and infill panels modeling approach, are referred to the aforementioned work.

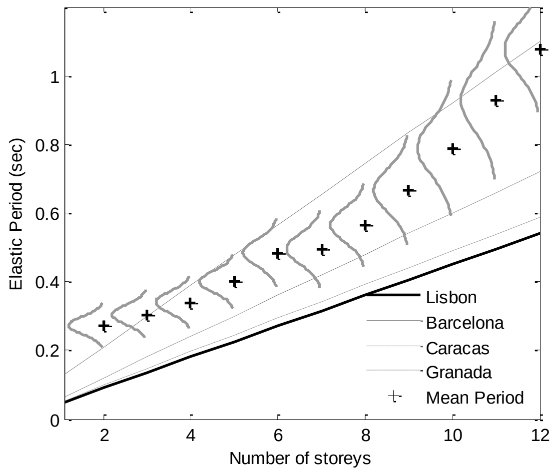

Figure 1. Sampled elastic periods of vibration of pre-code infilled frames and comparison with existing prediction for European cities.

RECORD SELECTION

In the context of fragility estimation, record-to-record variability is of upmost importance, as the variability in structural response, directly influenced by the latter, will have repercussions in the estimated probabilities of exceedance of the interested limit states (e.g. Lin *et al*., 2012). Amongst the available ground motion selection procedures, the Conditional Spectrum (CS) initially proposed by Baker and further developed by Jayaram *et al*. (2011), incorporating target mean and variance, provides an adequate mechanism for such estimations, conditioned to different levels of ground motion intensity. However, an important limitation of the latter is the fact that only characteristics of ground motion represented in terms of spectral accelerations can be considered. Thus, the General Conditional Intensity Measure (GCIM) approach proposed by Bradley (2010a) is adopted herein, whereby all the intensity measures identified as important to the outcome of the present exercise are foreseen. Further constraints for its selection are subsequently presented, according to the limitations of the referred methodology.

As enunciated by Bradley (2010a), the fundamental basis of the conditional response spectrum, according to which spectral accelerations can be assumed to have a multivariate lognormal distribution (Jayaram and Baker, 2008), is extended to any ground motion measure parameter of interest. It is thus proposed that for a given earthquake scenario, or rupture – *Rup* – any arbitrary vector of ground motion intensity measures has a multivariate lognormal distribution, to what follows that the distribution of *IM* given *Rup* (*IM*|*Rup*) conditioned on the occurrence of a particular *IMj* level presents identical statistical properties. In brief, since additional details shall be referred to Bradley (2010a), upon definition of appropriate Ground Motion Prediction Equations (GMPE) and correlation structure between the different *IMi* in *IM*, the conditional distribution of *IMi* given *IMj = imj* is obtained via total probability theorem from:

$$f_{IMi|IMj}(im_i|im_j) = \sum_{k=1}^{N_{Rup}} f_{IMi|Rup,IMj}(im_i|rup_k, im_j)P_{Rup|IMj}(rup_k|im_j)$$

Where $f_{IMi|IMj}(im_i|im_j)$ is the probability density function – *pdf* - of *IMi* given *IMj=imj*; $f_{IMi|Rup,IMj}(im_i|rup_k, im_j)$ is the *pdf* of *IMi* given *IMj=imj* and $Rup=rup_k$; and $P_{Rup|IMj}(rup_k|im_j)$ is the contribution weight of $Rup=rup_k$, determined through seismic hazard disaggregation. From the assumption that *IM* is characterized by a multivariate lognormal distribution, it follows that for each *IMi* in *IM*, $f_{IMi|Rup,IMj}(im_i|rup_k, im_j)$ has a univariate lognormal distribution, which can be defined by its conditional mean and standard deviation parameters:

$$\mu_{lnIM_i|Rup,IM_j}(rup_k, im_j) = \mu_{lnIM_i|Rup(rup_k)} + \sigma_{lnIM_i|Rup(rup_k)}\rho_{lnIM_i,lnIM_j}\varepsilon_{lnIM_j}$$

$$\sigma_{lnIM_i|Rup,IM_j}(rup_k, im_j) = \sigma_{lnIM_i|Rup}(rup_k)\sqrt{1-\rho^2_{lnIM_i,lnIM_j}}$$

determined as a function of *epsilon*, the number of standard deviations - $\sigma_{lnIM_j|Rup}(rup_k)$ - by which the logarithm of $IMj=im_j$ differs from the mean prediction of a particular GMPE - $\mu_{lnIM_j|Rup}(rup_k)$ – for a given rupture scenario – $Rup=rup_k$:

$$\varepsilon_{lnIMj} = \frac{lnIM_j - \mu_{lnIM_j|Rup}(rup_k)}{\sigma_{lnIM_j|Rup}(rup_k)}$$

Probabilistic Seismic Hazard Analysis and Disaggregation

Typically, causal earthquake magnitude, source-to-site distance, local site and fault properties are considered in the definition of different scenarios – *Rup* – contributing to hazard in a given site, as established by disaggregation (Bazzurro and Cornell, 1999). However, following the contributions of Lin *et al*. (2012) to the CS framework, seismic hazard disaggregation is evaluated in terms of Magnitude (M), Distance (R) and chosen Ground Motion Prediction Equations (GMPE), in order to ensure the consistency between target distributions of *IM* and ground motion properties expected at the site. Thus, $f_{IMi|IMj}(im_i|im_j)$ presented above are estimated as the sum of all the N_{Rup} contributing scenarios and N_{GMPE} considered: Atkinson and Boore (2006) and Akkar and Bommer (2010), following the work of Silva *et al.* (2014b) regarding hazard and risk assessment for mainland Portugal:

$$f_{IM_i|IM_j}(im_i|im_j) = \sum_{m=1}^{N_{GMPE}}\sum_{k=1}^{N_{Rup}} f_{IM_i|Rup,IM_j}(im_i|rup_k, im_j, GMPE_m)P_{Rup,GMPE_m|IMj}(rup_k, GMPE_m|im_j)$$

The Openquake engine (Monelli *et al*., 2012), which has been used for probabilistic seismic hazard analysis, does not currently address 3D disaggregation on M, R and GMPE; however due to its open-source nature, it was possible to produce the necessary intermediate results for the computation of $P_{Rup,GMPE_m|IM_j}(rup_k, GMPE_m|im_j)$, as described herein:

$$P_{Rup,GMPE_m|IM_j}(rup_k, GMPE_m|im_j) = \frac{v(IM_j, Rup|GMPE_m).P(GMPE_m)}{v(IMj)}$$

$P(GMPE_m)$ stands for the logic-tree weight assigned to $GMPE_m$; $v(IMj, Rup|GMPE_m)$ is the rate corresponding to the conditional probability of *IMj=imj*, using $GMPE_m$, assuming a Poissonian process; and $v(IMj)$ is the rate of occurrence of *IMj=imj*, computed from the correspondent rate of exceedance, as established by Bradley (2010a).

Record Database

Only three seismic events with significant ground motion were ever recorded in Portugal. For this reason, in order to create a sufficiently large database of candidates for selection, records from other regions in the world with similar geological and tectonic characteristics were gathered (e.g. Spain, France, Switzerland, and East United States). Properties of stable continent and active shallow crustal regions and corresponding faults influencing seismic hazard were respected to the maximum extent, with regard to referred information provided by Vilanova and Fonseca (2007) and Sousa and Campos-Costa (2009).

Nine hundred and eleven non-pulse ground motions and respective horizontal orthogonal components were selected from PEER [1] and ESMD [2] databases, with 815 and 96 records, respectively. Unarguably, the number of selected records from PEER database might concern to considerations on its influence in the final result. However, such consideration has been adopted due to the underlying advantage of the employed record selection procedure, which guarantees the consistency between the selected suits of natural ground motion records and the distribution of IMs expected at the interested site.

Selected IMs and ground Motion input

There is a large number of ground motion intensity measures proposed in the literature and, theoretically, any of such indicators can be considered in the GCIM selection approach. However, as further discussed, the latter hinges a number of constraints that, in practice, limit to the present date the number of IMs to be considered: a) GMPEs must be available for predicting marginal mean and standard deviation of the logarithm of each *IMi* in *IM*; and b) prediction of correlation between each *IMi* and *IMj* must be possible (Bradley, 2012a).

Following the findings from Sousa *et al*. (2014) regarding *efficiency* (Shome and Cornell, 1999) of a large number of IMs in the context of fragility assessment in which similar numerical models are analyzed, requirements deemed for the selection of IMs are considered herein. In order to ensure the applicability of ground motion prediction models to the specific case of mainland Portugal, preference is given to IMs that can either directly be predicted by the aforementioned GMPEs considered for spectral acceleration, or whose marginal mean and logarithmic standard deviation can be inferred from analogous statistical parameters provided for distribution of spectral ordinates, as enunciated by Bradley (2010b) and Bradley *et al*. (2009b). Target probabilistic distributions of *IMi* are thus determined for peak ground acceleration, PGA, peak ground velocity, PGV, acceleration spectrum intensity, ASI, Housner intensity, HI, and spectral ordinates within the range of 0.05 to 3.0 seconds; conditioned on *IMj* being the spectral acceleration at the assessed structures' fundamental period of vibration – $Sa(T_1)$ - for levels of 0.1g to 1.0g (with 0.1g intervals). The correlation models used herein are summarized in Table 1.

Table 1. Considered Correlation models between IMs

	$SA(T_i)$	PGA	PGV	ASI	HI
$SA(T_i)$	Baker and Jayaram (2008)	Baker (2007)	Bradley (2012b)	Bradley (2011)	Bradley (2011)
PGA	-	-	Bradley (2012b)	Bradley (2011)	Bradley (2011)
PGV	-	-	-	Bradley (2012b)	Bradley (2012b)
ASI	-	-	-	-	Bradley (2011)
HI	-	-	-	-	-

Record selection has been performed based on arbitrary component ground motion parameters, which enhances the available candidates in a 2D environment. However, in order to guarantee the consistency between the definition of intensity (geometric mean) and the selection and scaling procedure, the unconditioned standard deviations of different IMs, as determined by each GMPE, are corrected according to the proposal of Baker and Cornell (2006). For the case of PGA and PGV, for which correlation between arbitrary component ground motion in different directions is not provided by the latter study, the predicted relationship between standard deviation of arbitrary and geometric mean ground motion parameters estimated by Campbell and Bozorgnia (2008) is applied, as suggested by Baker (2005).

The number of ground motion records required to establish stable distributions of structural response is still not a straightforward matter in the present context. A number in the order of forty is proposed in the literature as a proxy for an adequate estimate, as recognized by Haselton *et al.* (2012); however, such number is highly dependent on the parameters used to characterize response, as well as the structural properties itself; which are herein consider following a probabilistic distribution rather than deterministic. For this reason, with due reference to the need of further investigation, 60 ground motion records are selected for each conditioning intensity level and each structure (two, five and eight story), as illustrated in figure 2 for the case of two story frames.

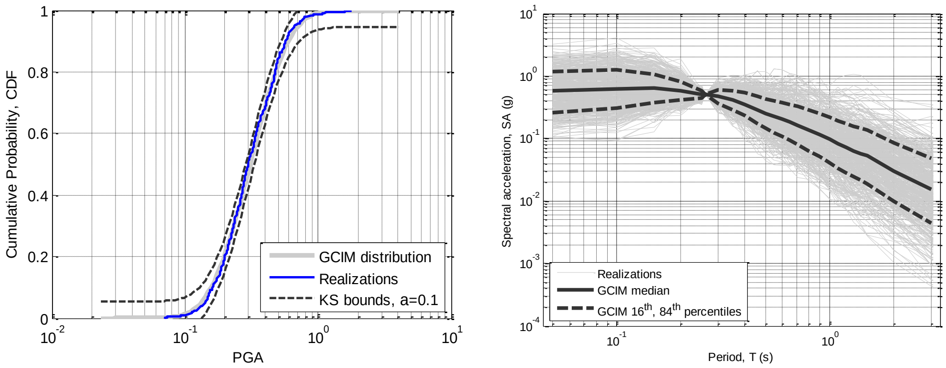

Figure 2. Target distributions and selected record sets for two story frames (PGA and spectral accelerations) conditioned on $Sa(T_1)$=0.5g.

FRAGILITY ASSESSMENT

As discussed in Silva *et al.* (2014a), the use of local criterion to define limit states when generating fragility curves for population of buildings may not be appropriate. Hence, global parameters are used in this study, considering maximum global drift (*GD*) criteria to the definition of *Slight Damage (SD), Moderate Damage (MD), Extensive Damage (ED)* and *Collapse (Col)* damage states.

Fragility functions are initially derived as a function of the primary intensity measure considered for record selection - $Sa(T_1)$ - so as to conduct an evaluation of its *efficiency*, which is the ability to predict the response of a structure with comparatively small heterogeneity (Shome and Cornell, 1999), as illustrated in figure 3, for limit states of *MD* and *Col* (2, 5 and 8 story buildings). To this end, the maximum likelihood estimation algorithm is used to fit probabilistic lognormal distribution curves, since it allows taking into account the non-constant variance of the observed damage exceedance probabilities.

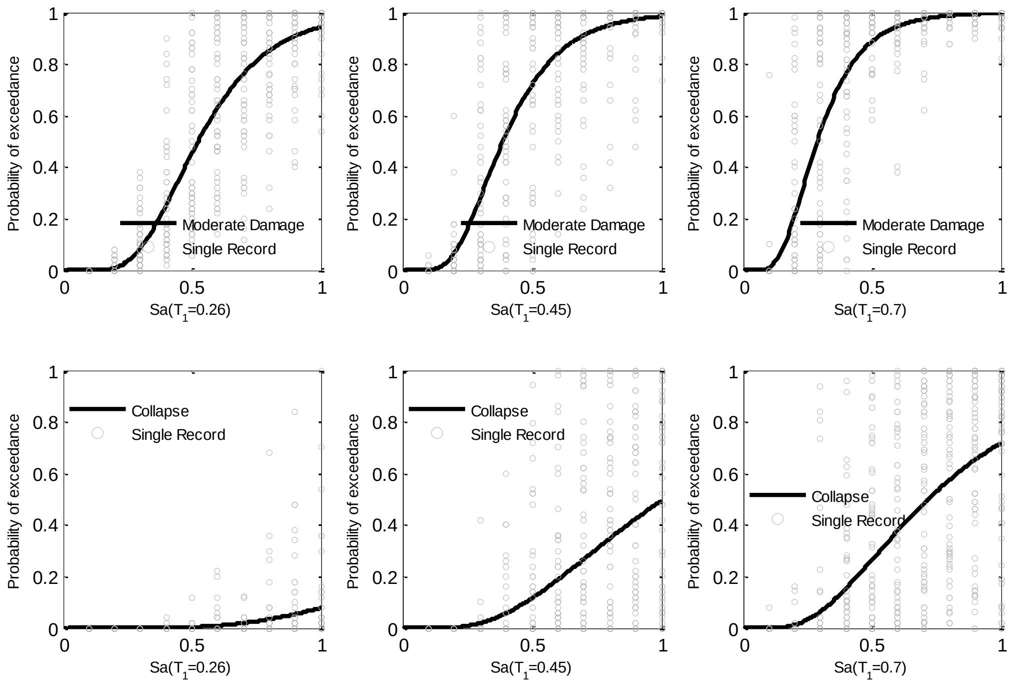

Figure 3. Fragility curves for limit states of *MD* and *Col* as a function of $Sa(T_1)$ – 2 (left), 5 (middle) and 8(right) story buildings.

It is possible to depict a significantly large dispersion in the sampled frames' response to each individual record, especially for limit state of Collapse. However, the latter can be explained by the employed hazard consistent record selection procedure, according to which a very significant dispersion in seismic intensity is expected for spectral ordinates in the elongated period range, as well as in the distribution of *IMs* other than $Sa(T_i)$, conditioned on $Sa(T_1)$ – figure 2. As further depicted in figure 6, $Sa(2.0*T_1)$ and *HI* systematically present the higher correlation with damage for the limit state of *Collapse* (and *Extensive Damage*, although not illustrated), which provides a clear indication on the level of period elongation experienced. If, for the sake of simplicity, one chooses to consider only spectral ordinates as the *IMs* governing response, it is possible to verify that the level of variability depicted in damage exceedance probabilities for each record (figure 3) is consistent with the dispersion of spectral acceleration values for periods higher than $2.0T_1$. In fact, as illustrated in figure 2 for the case of 2-story frames, values of $Sa(2.0*T_1)$ vary from a range of 1/10 to 2 times $Sa(T_1)$, which has been verified for all the assessed frame typologies, and furthermore in agreement with the levels of variability presented in the literature for similar record selection frameworks (e.g. Lin *et al*., 2012).

Multi-IM based formulation

By recognizing that $Sa(T_1)$, or any other IM for that matter, cannot comprehensively represent the effect of all the ground motion characteristics influencing structural response conditioned to a particular intensity level, one can further devise a methodology for assessing the contribution of each *IMi* in *IM* to the depicted response variability.

The variability in the probabilities of exceedance of different damage states - P_{LSi} - conditioned on increasing levels of $Sa(T_1)$ can be associated with *IM*, the vector of ground motion characterizing parameters - *IMi* - of particular relevance to the assessed seismic response problem. In this context, the previously illustrated dispersion in P_{LSi} given $Sa(T_1)$ (more generally, $f_{LSi|IM_j}$), is translated as the convolution of various uncertainties, as follows:

$$f_{LSi|IM_j}(ls_i, im_j) = \int_{IM} f_{LSi|IM,IM_j}(im, im_j) \, . \, f_{IM,IM_j}(im, im_j) . \, dIM$$

In which $f_{LSi|IM,IM_j}(im_i, im_j)$ and $f_{IM,IM_j}(im, im_j)$ are random variables representing, respectively: the distribution of exceedance probability of a particular limit state – LS_i – as a function of *IM|IMj*, the vector of all the considered *IMi* conditioned on *IMj=imj*; and the hazard consistent probabilistic distribution of *IM|IMj=imj*. By considering the procedure illustrated in figures 4 and 5, $f_{LSi|IMi,IM_j}(im_i, im_j)$ are determined as the conditioned lognormal fragility curves fitted to data pertaining to specific levels of $Sa(T_1)$ – *IMj=imj* in the general terminology - as a function of each *IMi|IMj* in *IM|IMj* (Sousa *et al.*, 2014). For the sake of synthesis, calculations are illustrated for *IMi|IMj* of *PGV, HI,* and $Sa(2.0*T_1)$; and limit states of *MD* and *Col*, for 5, and 8 story frames; conditioned on $Sa(T_1)=0.50g$.

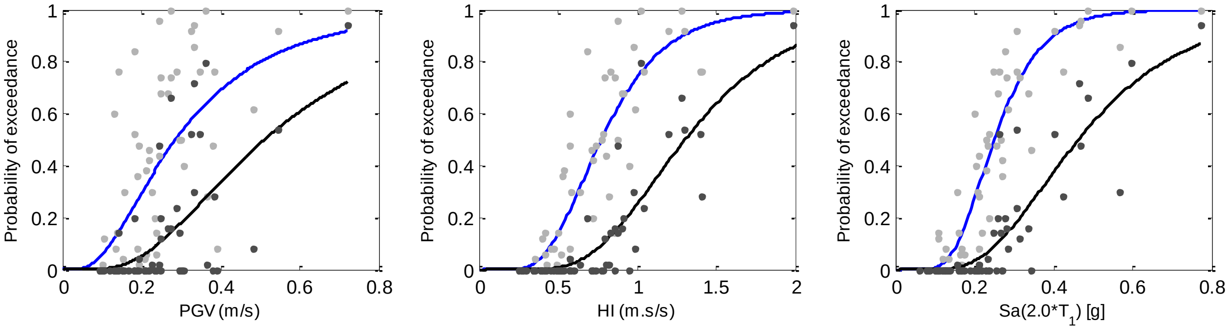

Figure 4. Lognormal fragility functions for 5 story frames - $f_{LSi|IMi,IM_j}(im_i, im_j)$; conditioned on $Sa(T_1)=0.50$; limit states of *MD* (blue) and *Col* (black).

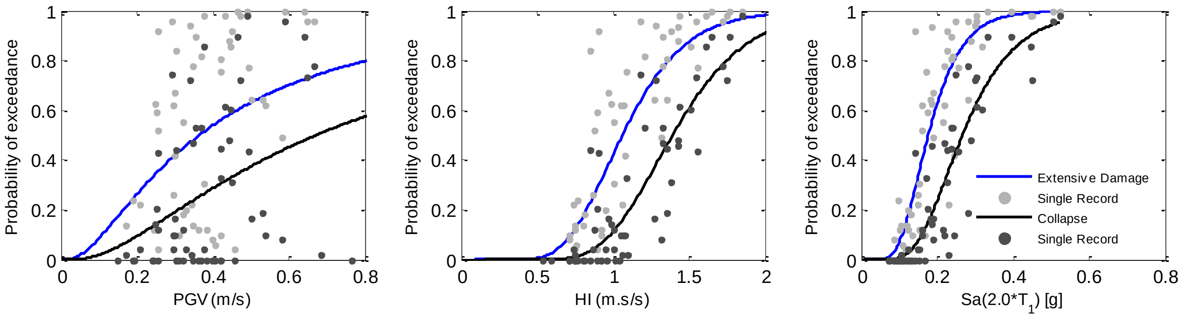

Figure 5. Lognormal fragility functions for 5 and 8 story frames - $f_{LSi|IMi,IMj}(im_i, im_j)$; conditioned on *Sa(T₁)=0.50g;* limit states of *MD* (blue) and *Col* (black).

Figure 6 illustrates the distinct level of dependence of structural response on different intensity parameters along the range of primary seismic intensity. In fact, follows from the formulation above that only the *IMs* in *IM* for which the conditional lognormal fragility curves' regression is statistically meaningful will affect the distribution of $P_{LSi|IM_j}(ls_i, im_j)$.

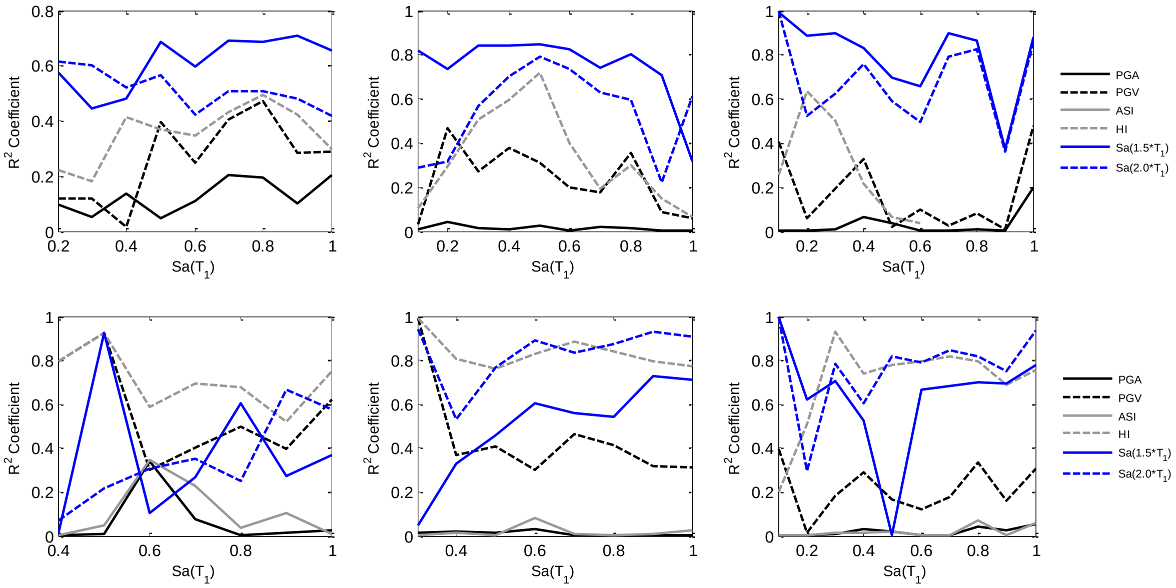

Figure 6. R^2 regression coefficient for *PGA, PGV, ASI, HI, Sa(1.50*T₁)* and *Sa(2.0*T₁)* for limit states of *MD* and *Col*; 2 (left), 5 (middle) and 8(right) story frames.

This can be apprehended by recognizing that for intensity measures for which the correlation with damage is statistically irrelevant, the bias induced in response conditioned on *IMj=imj* is not influenced by it. Since a single *IMi* tends to perform more satisfactorily (with less dispersion) in the prediction of damage for each level of $Sa(T_1)$, and the correlation between each of the considered *IMi* is appropriately considered in record selection, one can further adopt the mild assumption (whose impact is the subject of ongoing research) that considering only the *IMi* that better correlates with damage for each level of *IMj* is a satisfactory assumption; which greatly simplifies the previously mentioned formulation, as follows:

$$f_{LSi|IM_j}(ls_i, im_j) = \int_{IMi} f_{LSi|IM_i,IM_j}(im_i, im_j) \,.\, f_{IM_i,IM_j}(im_i, im_j).\, dIMi$$

Because of the foreseen correlation structure between different *IMs* in *IM*, the distribution of damage exceedance probabilities conditioned on a particular level of *IMj* is assumed to implicitly account for the remaining intensity measure variables and hence the first and second moments of $f_{LSi|IMj}(ls_i, im_j)$ can be defined, as subsequently presented:

$$\mu_{f_{LSi|IMj}(ls_i,im_j)} = \sum_{\overline{im}_i} \mu_{f_{LSi|IMi,IMj}(\overline{im_i},im_j)} . P_{IMi,IMj}\left(\overline{im_i}, im_j\right)$$

$$\sigma^2{}_{f_{LSi|IMj}(ls_i,im_j)} = \sum_{\overline{im}_i} \left(\mu^2_{f_{LSi|IMi,IMj}(\overline{im_i},im_j)} + \sigma^2_{f_{LSi|IMi,IMj}(\overline{im_i},im_j)}\right) . P_{IMi,IMj}\left(\overline{im_i}, im_j\right) - \mu^2_{f_{LSi|IMj}(ls_i,im_j)}$$

In which $\overline{im_i}$ represents the central value of Δim_i, such as $f_{IMi,IMj}\left(\overline{im_i}, im_j\right). \Delta im_i$ is approximately equal to $P_{IMi,IMj}\left(\overline{im_i}, im_j\right)$: the discrete probability of $IMi = \overline{im_i}$; and, $\mu_{f_{LSi|IMi,IMj}(\overline{im_i},im_j)}$ and $\sigma^2_{f_{LSi|IMi,IMj}(\overline{im_i},im_j)}$ refer to, respectively, the mean prediction and associated variance of conditional damage exceedance probabilities for a particular level of $\overline{im_i}$, as determined by the fitted conditional fragility curves, as a function of *IMi|IMj*.

Following the study by Bradley (2010c) for the case of component fragility, dispersion in $f_{LSi|IM_iIM_j}\left(im_i, im_j\right)$ - $\sigma^2_{f_{LSi|IMi,IMj}(im_i,im_j)}$ - is found to vary along the range of *IMi|IMj* intensities, as depicted in figure 7, in which such variability is illustrated for conditional lognormal fragility curves as a function of *HI* and *Sa(2.0*T$_1$)* - limit states of *Col* – conditioned on *Sa(T$_1$)=0.5g.* A bootstrap sampling method with replacement (Wasserman, 2004) is used to determine the uncertainty in distribution of regression parameters, using 500 synthetic datasets randomly extracted from the original distributions of damage exceedance probabilities.

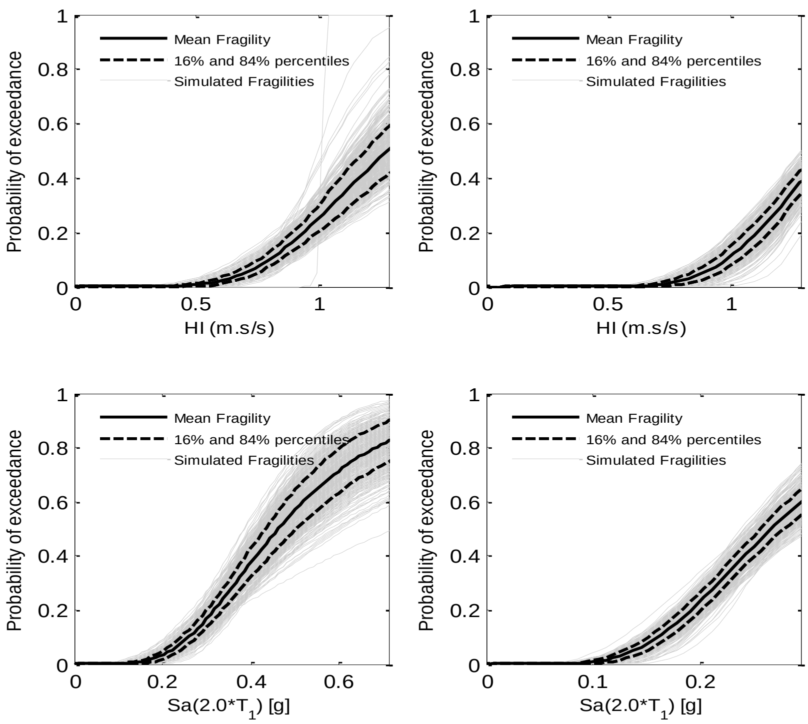

Figure 7. Individual, mean, 16% and 84% percentile fragilities - $f_{LSi|IM,IMj}\left(im_i, im_j\right)$ – for intensity measures of *HI* and *Sa(2.0*T$_1$)* - limit state of *Col* – conditioned on *Sa(T$_1$)=0.5g;* 5(left) and 8(right) story buildings.

RESULTS

For illustration purposes, the distribution of damage exceedance probabilities as a function of *IMj* - $f_{LSi|IMj}(ls_i, im_j)$ - here considered as *Sa(T₁)*, is determined for limit states of *MD* and *Col*, in order to outline the comparison between the mean and dispersion evidenced in figure 3, herein reproduced with additional results computed according to the proposed methodology.

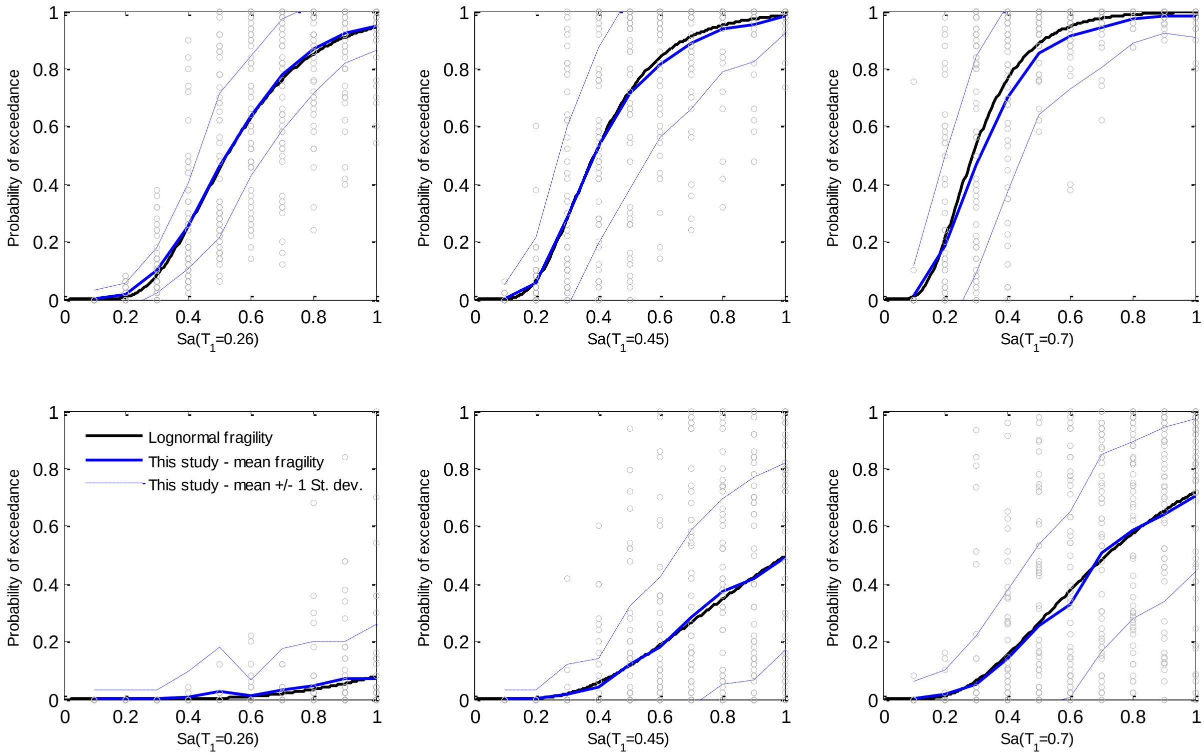

Figure 8. Mean, 16% and 84% percentiles of exceedance probabilities as a function of *Sa*(T_1); limit states of *SD* (upper) and *Col* (lower); 5 story buildings.

It is clear that, because record selection has been comprehensively performed for a set of intensity measures for which full correlation is accounted for, the small differences attained in mean prediction of building fragility arise from the fact that real target distribution of *IMi|IMj* - $f_{IMi,IMj}(imi, im_j)$ – is used as a way to correct the bias induced by sets of selected records for which the empirically derived distribution does not match the target in a statistically meaningful way. However, as clearly illustrated in figure 8, the greatest contribution of the presently proposed methodology consists on the appropriate treatment of uncertainty in damage exceedance proobabilities, particularly in what concerns consistency between seismic intensity and its hazard dependent record-to-record variability.

CONCLUSION AND FUTURE DEVELOPMENTS

By considering the effect of a set of *IMs* along the range of building response – characterized as a function of the commonly used *Sa*(T_1) - as well as the "weighted" contribution of each level of conditioned *IMi* according to both hazard consistency considerations and correlation with damage, the proposed methodology consists of a contribution to an appropriate treatment of the uncertainty arising from record-to-record and material / geometric variability in the context of fragility and vulnerability assessment of populations of buildings. Nonetheless, further developments are referred to: the study of a wider range of intensity measures; the evaluation and validation of theoretical assumptions outlined in the present manuscript; and, of particular relevance, the methodology repercussions on earthquake loss estimation of building portfolios.

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