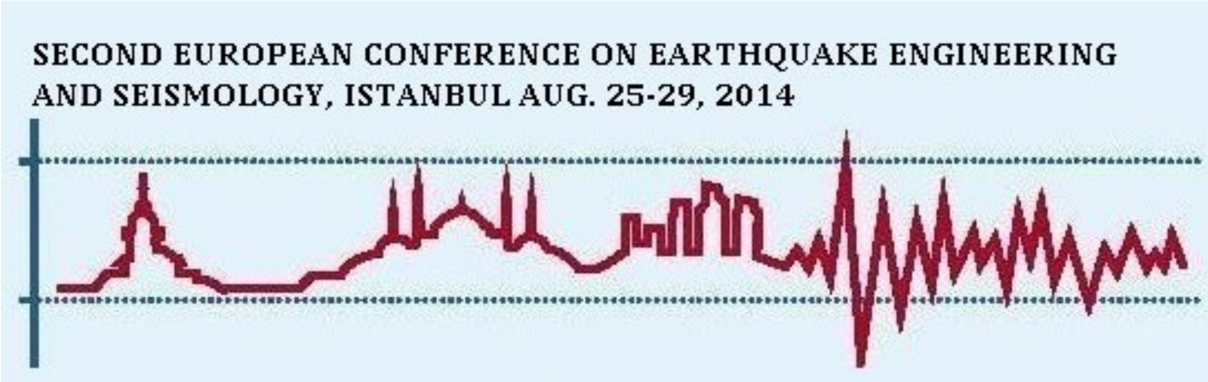

FRAGILITY CURVES FOR MASONRY BUILDINGS FROM EMPIRICAL AND ANALYTICAL MODELS

Serena CATTARI[1], Sergio LAGOMARSINO[2] and Daria OTTONELLI[3]

ABSTRACT

The general aim of paper, in the framework of seismic risk analysis, faces issues related to the tools for a reliable definition of the exposure and vulnerability models. The first one identifies in general people, buildings, infrastructure, or other elements present in hazard zones that are thereby subjected to potential losses. The second one allows determining the probabilistic distribution of loss ratio given a certain level of seismic demand passing through the concept of fragility curves, in turn representing the damage state exceedance probabilities.

In this paper, for the exposure model, the focus is on the description of a suitable taxonomy for ordinary masonry buildings and its use in the assets classification. Instead, for what concern the vulnerability models, a procedure, from both expert elicitation and analytical models for estimating the fragility curves, is described. Moreover, both methodologies are performed on an example of application, in order to use the expert elicitation curves to validate the analytical ones.

INTRODUCTION

Seismic risk is defined as the potential of negative consequences of hazardous events that may occur in a specific area unit and period of time. The outcome is the mean annual rate of fixed consequences, which is obtained by the probabilistic convolution of hazard, vulnerability and exposure. The paper is mainly focused on the exposure and vulnerability components.

For what concern the exposure, the paper refers only to the ordinary unreinforced masonry buildings, recalling the need of a suitable taxonomy as preliminary step to collect all necessary data of the stock examined and then to group the assets in classes characterized by a homogeneous behavior. It is important to note that this step is particularly important since the definition of the classes to be considered also influences the variability of the fragility curves.

Concerning the vulnerability, it represents the probability of a certain asset to suffer loss due to the effects of a ground shaking passing through the definition of fragility curves. Fragility is defined as the probability of exceeding a limit states, given a range of intensity measure, and it can derive from different approaches (e.g. as recently discussed in Rossetto et al., 2014): 1) empirical; 2) expert elicitation based; 3) analytical, based on nonlinear static approaches through simplified and detailed models, or based on linear dynamic approaches; 4) hybrid methods. Among these, the paper is concentrated on the expert elicitation based models and their adoption to complete and validate the analytical ones. In fact, although the use of analytical model has the advantage to keep explicitly into account the various parameters that determine the structural response, at the same time the reliability of the vulnerability assessment is affected by the actual capability of the model to simulate the seismic

[1] Assistant Professor, DICCA - University of Genoa, Genoa (Italy), serena.cattari@unige.it
[2] Full Professor, DICCA - University of Genoa, Genoa (Italy), sergio.lagomarsino@unige.it
[3] Phd student, DICCA - University of Genoa, Genoa (Italy), daria.ottonelli@unige.it

response of the class under investigation. On the other hand, the expert elicitation based models are obtained from observed damage and directly correlated to the actual seismic behavior of buildings: thus, they may constitute the fundamental support to validate and corroborate the use of the analytical ones. In the paper the reference for the expert elicitation based model is the Macroseismic Model proposed by Giovinazzi and Lagomarsino (2006), vice versa for the analytical approach is the DBV-*masonry* Model (Lagomarsino and Cattari, 2013), applied and validated in different case study as described in (Cattari et al., 2010, Cattari et al., 2013, Lagomarsino and Cattari, 2014). In particular, for both, a procedure to derive fragility functions for masonry structures is proposed.

Indeed, in a seismic risk analysis, fragılıty functıons can be defined by using existing ones. In fact, in literature many curves have been developed for masonry buildings (Benedetti et al. 1988, Bernardini et al. 1990, HAZUS 1999, Kappos et al. 2008, D'Ayala and Ansal 2009, Barbat et al. 2010, Cattari et al. 2010, Cattari et al. 2013, D'Ayala and Paganoni 2010, Ferreira et al. 2013), but the use of existing fragility functions has to be made carefully and in some cases is questionable, since, for example, the empirical fragility functions are strongly influenced by the reliability of the damage assessment (often made by a quick survey aimed to other scopes, as the building tagging for use and occupancy) and also, in case of masonry buildings the structural characteristics are dependent from the local seismic culture and the available materials in the area. So, in order to overcome these disadvantages, the present paper is framed in the studies aimed to propose methodologies to derive new fragility functions.

In particular, the following chapters deal with: i) the role of the taxonomy and classification of buildings, in a seismic risk analysis; ii) the definition of the key assumptions, treatment of uncertainties and modeling issues, adopted for the derivation of fragility functions from both expert elicitation and analytical models; iii) the application of the two different vulnerability models for different classes of buildings, in order to derive reliable fragility curves, compare the results and define the variability of the coefficient dispersion according to the definition of a proper classification develop by a taxonomy.

THE ROLE OF TAXONOMY AND CLASSIFICATION FOR THE EXPOSURE MODEL

The vulnerability assessment at territorial scale requires, first of all, to group the buildings of the exposure, that have a similar seismic behavior, in order to evaluate the damage and losses of the built environment due to a given hazard assessment. To this aim, a proper taxonomy can be used in order to classify the buildings and then select the classes which the computation of fragility functions must be addressed to.

The exposure model, object of the analysis, consists of masonry buildings. These latter constitute a wide variety of constructions, which are characterized by very different types of masonry and structural systems, moving through historical periods and geographical areas. With reference to the structural systems, ancient constructions, but also recent vernacular ones, are very different from engineered masonry buildings, such as confined or reinforced masonry. The former were built by an empirical approach and are usually vulnerable, first of all to local mechanisms (out-of plane behavior); however, in high seismic areas specific details were adopted to prevent from damage (metallic tie rods, timber belts, buttresses, connections of horizontal diaphragms to masonry walls, etc.). The latter have been specifically conceived to withstand the earthquake, after a detailed damage observation, as in the case of confined masonry (widely adopted in South American countries), or on the base of modeling and capacity design criteria, as in the case of unreinforced masonry building (with reinforced concrete - RC - ring beams at floor level) or reinforced masonry.

Among the masonry building may also be considered the mixed structures, such as the traditional mixed masonry-timber buildings or the rather modern mixed masonry-RC buildings.

In the following, the main features that are useful for the taxonomy of masonry buildings are listed according to the general approach proposed in SYNER-G project (http:// www.vce.at/SYNER-G/). The main categories are: FRM – Force Resisting Mechanism; FRMM – Force Resisting Mechanism Material; P – Plan; E – Elevation; CO – Cladding & Openings; DM – Detailing & Maintenance; FS – Floor System; RS – Roof System; HL – Height Level; CL – Code Level. Within each category, the list of possible options is defined by proper acronyms and described in detailed in Lagomarsino and Cattari, 2014. Naturally other taxonomy proposals are present in literature for example in HAZUS (FEMA-443, 2003) or the newest GEM Building Taxonomy (Brzev et al., 2012).

It is important to note, that in the case of masonry buildings the FRM is always the Bearing Walls system (BW), which can present very different seismic behavior depending on geometry and constructive details. Usually reference is made to Out-of-Plane (OP) and In-Plane (IP) mechanism, depending on the connections and distance between masonry walls, as well as on the stiffness of horizontal diaphragms. If a global seismic (box-type) behavior can be assumed, a sub-classification is possible: each single wall may be analyzed by an equivalent frame model (EF) or by simplified models that assume the hypotheses of strong (SSWP) or weak (WSSP) spandrels. The choice of the most reliable model depends on available as-built information.

The category FRMM considers different structural material: Unreinforced Masonry (URM); Reinforced Masonry (RM); Confined Masonry (CM); Timber-framed Masonry (TM); mixed Masonry-RC (MRC). In particular, in the URM case, a detailed classification is important, with reference to blocks and mortar characteristics, because the mechanical properties vary in a wide range.

The configuration of the building Plan (P) is very important for the seismic vulnerability, both with reference to the regularity (R, IR) and to the possible interaction with other buildings (Isolated – I – or Aggregated in urban blocks – A). This information is useful to address the most probable collapse mechanisms (BW classification).

Information on the regularity in Elevation (E) may help in the definition of the behavior factor and the ductility, due to the possible different localization of the weak story.

The role of non-structural elements is almost negligible in masonry buildings, but it is important to know the regular distribution and percentage of openings (CO). A regular distribution (RO) may promote the WSSP behavior, which is characterized by a higher displacement capacity but a lower strength than the SSWP case. Moreover, a High percentage of openings (H%) at the base story, typical in the case of shops, may produce a weak story mechanism, which has a low displacement capacity.

Another important category, in particular in the case of URM buildings, is the quality of constructive details and the state of maintenance, which is an essential prerequisite in order to exploit the former aspect (DM). The attribution of High Quality Details (HQD) must consider the adherence to the rules of the art, which altogether define a local code of practice referred to different scales of the construction: the masonry (way to assure interlocking and transversal connection), the wall (distribution of openings, lintels, etc.) and the global construction (wall-wall and wall-horizontal diaphragms connections). The systematic presence of effective tie rods (WT) or ring beams (WRB) may prevent from out-of-plane mechanisms and increase the strength and ductility of spandrels, for the in-plane behavior; it is worth noting that RC ring beams drive the seismic response to weak story mechanism (SSWP behavior), while tie rods increase the ductility of uniform mechanisms (WSSP behavior).

The Floor System (FS) influences the seismic behavior, with reference both to its mass (which increases the horizontal seismic actions) and its stiffness (which allows a certain degree of redistribution of the horizontal seismic actions between the vertical walls). A rough categorization is obtained by distinguishing between Rigid (R) and Flexible (F); the attribution has to consider not only the stiffness but also the effectiveness of the connection with vertical walls. A more detailed classification can consider also the material and configuration (i.e. the presence of masonry vaults can also induce horizontal thrusts).

Similar information are required on the Roof System (RS), which is an important parameter for the vulnerability assessment, because of its mass (dynamically amplified due to its position at the top of the building) and the possible presence of a horizontal thrust (Tr), which can induce local collapse mechanisms.

The Height Level (HL) is very important because it influences very much the seismic vulnerability and is always available or very easily detectable. The possible categories (L, M, H and Ta) must be redefined, in terms of number of stories, for masonry buildings, because they are on average lower than RC or steel buildings.

Finally, the Code Level (CL) category is very important and must be properly defined in the case of masonry buildings, which are usually old and not seismically designed (PC); in this case, it is useful to estimate the local seismic culture, which is high (HAC) in areas frequently affected by earthquakes. For modern buildings, designed by considering a seismic code (LC, MC and HC), the categories should mainly consider the seismic hazard used for the design, taking also into account the accuracy of the code provisions.

Once the taxonomy as reference is defined, depending on the available data and after a preliminary study of the characteristics of the built environment in the urban area under investigation, the first step of the vulnerability assessment is to proceed to a proper classification of buildings. To this aim, among the available information, the parameters that mostly affect the seismic behavior must be singled out. Each vulnerability class, which can be synthetically named by a number or a short acronym (Class 1: /BW/URM-FB-HM/R/R/RO/HM/R/P/M/PC-MAC/, …), is clearly identified by a precise taxonomy, that is a list of category and related classification information. Missing information in the taxonomy means that no data are available to better describe the buildings, so fragility functions must represent the average vulnerability of a large set of configurations. On the contrary, if some parameter is excluded, all other options should be listed in the taxonomy.

Naturally, in principle fragility functions must be defined for each building class. Despite this, it is necessary to properly balance the effort in analyzing several classes and the actual possibility of acquiring a large amount of data with the actual reliability of results achieved. In fact, it is worth noting that, when few building classes are used, each one including constructions characterized by quite different behavior, the dispersion usually increases; on the contrary, a too much detailed classification could lead to the definition of classes with quite similar fragility functions, even if with a lower dispersion.

COMPUTATION OF FRAGILITY CURVES FROM BOTH EXPERT ELICITATION AND ANALYTICAL MODELS

The fragility function gives the probability that a generic Limit State *(LS*) is reached given a value im of the Intensity Measure *IM*; generally they can be expressed as:

$$P(LS \geq ls_i | IM) \tag{1}$$

where ls_i is a particular predefined state of damage.

In the case of analytical methods, referring in particular to the DBV-*masonry* Model, proposed in Lagomarsino and Cattari (2013), the seismic vulnerability of an assets class is described through the definition of geometrical and mechanical characteristics, that allow to relate the structural response to the seismic action (related to instrumental *IMs*) and, consequently, the damage effects. For these models, the fragility curves are commonly developed on the basis of a lognormal distribution, as follows:

$$p_{LS}(im) = P(d > D_{LS}|im) = P(im_{LS} < im) = \Phi\left(\frac{log\left(\frac{im}{IM_{LS}}\right)}{\beta_{LS}}\right) \tag{2}$$

Where: d is a displacement representative of the building seismic behavior (that is the Engineering Demand Parameter -EDP adopted in case of the analytical method), D_{LS} is its Limit State threshold, IM_{LS} is the median value of the lognormal distribution of the intensity measure im_{LS} that produces the LS threshold and β_{LS} is the dispersion.

A fragility function is thus defined by two parameters: IM_{LS} (considered representative of the average seismic behavior of buildings of a particular class) and β_{LS} (relative to the uncertainty).

The dispersion β_{LS} depends on different contributions, related to: a) the uncertainties in the seismic demand (epistemic β_H, for the derivation of the hazard curve, and intrinsic β_D, in the variability of the seismic input described only by the value of *IM*); b) the uncertain definition of the Limit State threshold (β_T); c) the variability of the capacity (β_C) of buildings that belong to the considered vulnerability class (which collects buildings of different behavior, even if characterized by the same taxonomy tags). As in general all the above contributions can be assumed statistically independent, the dispersion is given by:

$$\beta_{LS} = \sqrt{\beta_H^2 + \beta_D^2 + \beta_T^2 + \beta_C^2} \tag{3}$$

In the case of analytical methods, the value IM_{LS} that produces any LS threshold can be obtained by the following steps:

- the definition of the seismic vulnerability of the building described by its capacity curve (a bilinear curve without hardening), which gives the acceleration A of an equivalent nonlinear single-degree-of-freedom system, as a function of its displacement D;
- the definition of the seismic demand, expressed by an Acceleration-Displacement Response Spectrum (ADRS), which gives the spectral acceleration Sa as a function of the spectral displacement Sd, for a damping coefficient $\xi_0 = 5\%$, considered valid in the initial elastic range. Usually in hazard analysis the spectral shape is assumed constant with the annual rate of exceeding, which is given by the hazard curve as a function of a proper IM of the ground motions;
- the computation of IM_{LS}. It is based on nonlinear static procedures based on the use of inelastic (e.g. according to the N2 Method proposed in Fajfar, 2000) or overdamped spectra (e.g. according to the Capacity Spectrum Method as originally proposed in Freeman, 1998). Following the approach based on the use of overdamped spectra (synthetically illustrated in Figure 1), once the thresholds D_{LS} have been fixed on the capacity curve and the corresponding equivalent viscous damping ξ_{LS} is evaluated, which also takes into account the hysteretic contribution, IM_{LS} is computed. Where, S_{d1} is the displacement response spectrum, normalized to IM, T_{LS} is the linear equivalent period corresponding to LS and $\eta(\xi_{LS})$ is the damping correction factor (CEN 2004):

$$IM_{LS} = \frac{D_{LS}}{S_{d1}(T_{LS})\eta(\xi_{LS})} \tag{4}$$

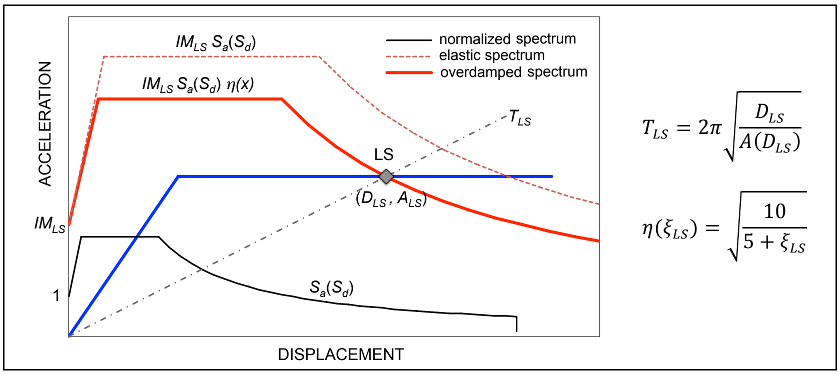

Figure 1. Application of overdamped spectra for the evaluation of IM_{LS} (Lagomarsino and Cattari, 2014)

In terms of total value of dispersion, literature (ATC-58, 2011, FEMA-NIBS, 2003 and Kappos and Panagopoulos, 2010) suggest a default value of 0.6, or different values of 0.75, 0.70, and 0.65 for buildings designed to old, moderate, and modern codes, respectively. In some case, studies that adopt analytical procedures numerically estimate just one component of the overall β_{LS} uncertainty, i.e. the record to record variability. Vice versa, in the procedure proposed, the dispersion β_{LS} is computed analyticall by considering the definition of each contribution. In particular in case of:

- β_D - *the uncertainty on the spectral shape*, which plays a significant role due to the large variability of possible records, it is necessary to define the response spectra $Sa_{1,16}(S_d)$ and $Sa_{1,84}(S_d)$, for the confidence levels 16% and 84%. Figure 2 shows a typical example of a median response spectrum and the corresponding confidence intervals, if the Peak Ground Acceleration (PGA) is used as IM. For each LS, the estimation of β_D requires the evaluation of the intensity measures $IM_{D,16}$ and $IM_{D,84}$ that correspond to a displacement demand equal to D_{LS}, on the median capacity curve of the considered class of buildings, by using the confidence levels response spectra $S_{a1,16}(S_d)$ and $S_{a1,84}(S_d)$ respectively. It results:

$$\beta_D = 0.5\left|log\left(IM_{D,84}\right) - log\left(IM_{D,16}\right)\right| \tag{5}$$

- β_H - *the epistemic uncertainties in the seismic sources and the attenuation laws* it gives rise to confidence intervals, which can be summarized by the hazard curves $\lambda_{16}(im)$ and $\lambda_{84}(im)$ representative of the confidence levels 16% and 84%. For each *LS*, it is necessary to valuate IM_{LS} that corresponds to the displacement demand D_{LS} on the median capacity curve of the considered building class, by using the median response spectrum $S_{a1}(S_d)$. The dispersion β_H is thus given by:

$$\beta_H = 0.5\left[log\left(IM_{H,84}[\lambda(IM_{LS})]\right) - log\left(IM_{H,16}[\lambda(IM_{LS})]\right)\right] \tag{6}$$

where $IM_{H,16}$ and $IM_{H,84}$ are the inverse functions of $\lambda_{16}(im)$ and $\lambda_{84}(im)$, respectively, and $\lambda(im)$ is the median hazard curve;
- β_C - *the dispersion on the capacity curve* of a masonry building it is related to random variables, such as the material parameters (strength and stiffness on masonry), the geometry (effective thickness of masonry walls and vaults), the drift capacity of masonry panels or the in-plane stiffness of horizontal diaphragms, but also to epistemic model uncertainties. From these parameters, an "equivalent capacity curve" must be evaluated representative of a wide class of buildings, defined by the taxonomy through a proper list of tags. Then the above parameters have to be considered as random variables, with dispersion compatible with the variability of the characteristics of buildings in the class. The uncertainty propagation can be evaluated by using the response surface method (Pagnini et al., 2011, Liel et al., 2009);
- β_T - *the uncertainty on the definition of the LS thresholds* it is also subjected to dispersion, because models adopted for the evaluation of the capacity curve are simplified and the displacements D_{LS} usually derives from a heuristic approach. Considering the median capacity curve, D_{LSk} (k=1,..4), very simple uniform distributions for each *LS* are suggested (Figure 2). For each *LS*, it is then necessary to evaluate $IM_{T,16}$ and $IM_{T,84}$ that correspond to a displacement demand equal to $D_{LS,16}$ and $D_{LS,84}$, respectively computed on the median capacity curve of the considered class of buildings by using the median response spectrum $S_{a1}(S_d)$. The dispersion β_T is given by:

$$\beta_T = 0.5\left[log\left(IM_{T,84}\right) - log\left(IM_{T,16}\right)\right] \tag{7}$$

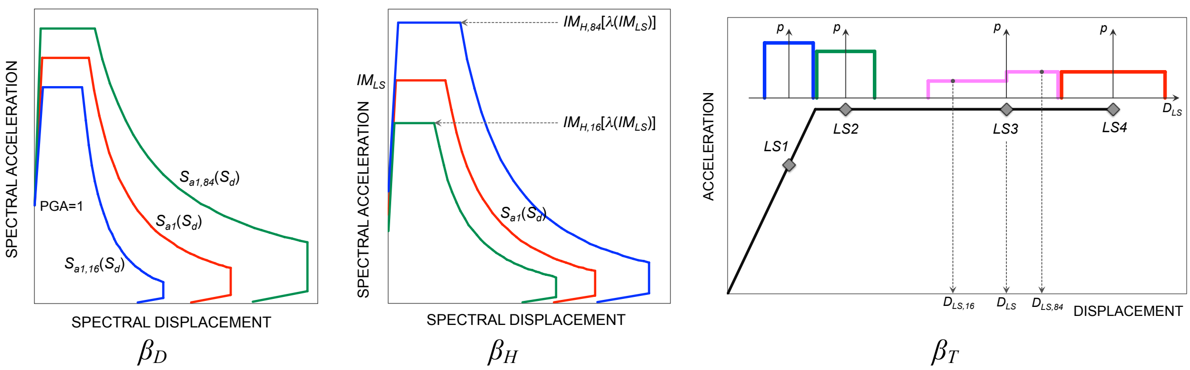

Figure 2. Uncertainty computed for the dispersion β_{LS} (Lagomarsino and Cattari, 2014)

Vice versa, in the case of the expert elicitation model adopted that refers to the macroseismic method of Giovinazzi and Lagomarsino (2006),directly derived from the European Macroseismic Scale (Grunthal et al., 1998) and based on an empirical Intensity Measure (i.e. *I* – the Macroseismic Intensity*)*, the fragility functions can be evaluated by the binomial distribution:

$$p_{LSk} = \sum_{i=k}^{5} p_{DSi} \quad (k = 1,..5) \tag{8}$$

$$p_{DSk} = \frac{5!}{k!\,(5-k)!}\left(\frac{\mu_D(I)}{5}\right)^k \left(1 - \frac{\mu_D(I)}{5}\right)^{5-k} \quad (k = 0,..5) \tag{9}$$

Where μ_D is the mean damage, as a function of the macroseismic intensity *I*, expressed by the vulnerability curves. Limit States can be identified on the vulnerability curve as points for which $p_{LSk}=0.5$ (k=1,..5). The vulnerability curve is, for the macroseismic method, analogous of the capacity curve for the analytical ones.

If a proper correlation law between intensity *I* and *PGA* is assumed, then the fragility functions may be converted in terms of PGA. To this aim, many correlations may be found in literature, which have been calibrated in different areas and are usually in the form:

$$I = a_1 + a_2 Log(PGA) \tag{10}$$

Figure 3 shows the result of such conversion from *I* to *PGA* based on both Faccioli and Cauzzi (2006) and Murphy and O'Brien (1977). Indeed, they looks very similar to a lognormal cumulative distribution (dashed lines in Figure 3) that may be traced by computing β_{LS} values aimed to fit the original curves.

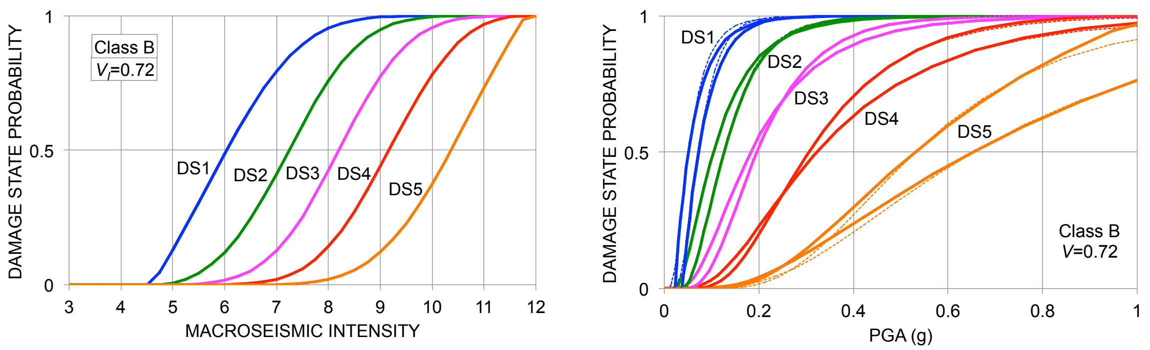

Figure 3. Fragility functions of in intensity and PGA (Lagomarsino and Cattari, 2014)

From the above-mentioned issues, related to the different approaches, it is evident as one of the crucial point in defining the fragility curves is the identification of a proper Intensity Measure (*IM*). It comes out from different constraints, which are first of all related to the adopted hazard model, to the typology of the exposed asset but also to the availability of data and fragility functions for all different exposed assets.

Correlation is necessary when hazard and vulnerability assessments are made by using different *IMs* or one wants to calibrate analytical fragility functions by available empirical fragility functions. Anyhow, the use of correlation always increases the uncertainties of the results (dispersion β_{LS} of the fragility function).

The seismic performance of a masonry building cannot be described by only one *IM* but, at least, the response spectra shape should be known. If a vector-valued hazard assessment is available (Bazzurro and Cornell, 2002), more than one *IM* could be used and vector-valued fragility functions derived (e.g. Gehl et al., 2013). If already available fragility functions are used, it is better to refer to the spectral value for the period compatible with the specific Limit State threshold (acceleration $S_a(T)$ and displacement $S_d(T)$ response spectra are linked by the period of vibration *T*, so the two *IMs* are equivalent). In this case the dispersion β_{LS} of the fragility function is mainly due to the variability of the capacity of buildings in the class.

Most of available fragility functions are in terms of *PGA*; in this case, if the difference between the spectral shapes of the input motion (obtained by the hazard assessment) and that used for deriving the fragility function is known, it is possible to properly tune the last one. Otherwise, the use of PGA as IM implies a wider dispersion β_{LS} of the fragility function, due to the uncertainty in the spectral shape.

As masonry buildings are usually not flexible, *PGD* or spectral values for long periods (*T>1 s*) are not significant, except for some types of monumental structures (churches, slender towers) or for the verification of local mechanisms.

With reference to the local site amplification, spectral values are better correlated with vulnerability, because they take into account the modification of the seismic input for the significant periods. If *PGA* is used, fragility functions should be tuned by considering a mean ratio between the spectral values on local site and stiff soil conditions, for the relevant periods of the buildings, or a greater value of the dispersion should be used, in order to consider the increased uncertainty due to the spectral demand (β_D).

In case of using empirical *IM* (macroseismic intensity), it is not correct to include local site amplification in the hazard curve, because this phenomena affects buildings depending on their dynamic properties; a possible solution is to modify the empirical fragility function, so considering it as representative of the vulnerability of a particular class of buildings on a specific soil type (Giovinazzi and Lagomarsino, 2006).

EXAMPLE OF APPLICATION OF ANALYTICAL AND EXPERT ELICITATION MODELS TO DERIVE FRAGILITY CURVES

In the following, the above-mentioned procedures were implemented in an application composed of different classes of buildings, in order to derive, compare and validate reliable fragility curves. In particular, four classes of URM building are considered, related to different types of Blocks (see Table 5.1 in Lagomarsino and Cattari, 2014, that illustrates in detail the taxonomy adopted) for the meaning of the taxonomy tags): rubble (HS-RU), uncut (HS-UC), fired bricks (FB), hollow clay tile (HC); lime mortar (LM) is considered for the first three block types, while cement mortar (CM) is assumed for the latter. Different classes of height have been considered, in order to show what a mechanical based model is able to distinguish in the seismic behavior, with reference to the four DSs. Finally, in case of FB, both the alterative cases of presence of tie rods (WT) or r.c. ring beams (WRB) are considered, as well as, in the former case, the influence of plan irregularity. Summing up, ten different classes have been investigated (Table 1).

Table 1. Vulnerability Classes considered in the analysis

Vulnerability Class	List of tag
URM1-L	BW-IP\URM-HS-RU-LM\R\R\x\LQD-WoT-WoRB\F-T\P-T\L\PC
URM2-L	BW-IP\URM-HS-UC-LM\R\R\x\LQD-WT\F-T\P-T\L\PC
URM2-M	BW-IP\URM-HS-UC-LM\R\R\x\LQD-WT\F-T\P-T\M\PC
URM3-M	BW-IP\URM-FB-LM\R\R\x\LQD-WT\R-S\P-RC\M\PC
URM3-H	BW-IP\URM-FB-LM\R\R\x\LQD-WT\R-S\P-RC\H\PC
URM3-M-IR	BW-IP\URM-FB-LM\IR\R\x\LQD-WT\R-S\P-RC\M\PC
URM3-H-IR	BW-IP\URM-FB-LM\IR\R\x\LQD-WT\R-S\P-RC\H\PC
URM4-M:	BW-IP\URM-FB-LM\R\R\x\HQD-WRB\R-RC\P-RC\M\PC
URM4-H	BW-IP\URM-FB-LM\R\R\x\HQD-WRB\R-RC\P-RC\H\PC
URM5-M	BW-IP\URM-HC-CM\R\R\x\HQD-WRB\R-RC\P-RC\M\MC

Firstly, the analytical formulation (DBV-*masonry* Model, Lagomarsino and Cattari, 2013) makes reference to the Strong Spandrels Weak Piers (SSWP) model, which corresponds to the shear-type frame model and is associated to the occurrence of a soft-story failure, under the simplified hypothesis, in the evaluation of the total base shear, that all masonry piers fail at the same time, which is true if they are more or less of the same size and the building is regular in plan. Starting from these assumptions, the vulnerability of actual buildings, which do not meet the above mentioned hypotheses, is estimated applying proper corrective factors: thus, it is possible to evaluate the capacity curve of buildings characterized by Equivalent Frame (EF) or Weak Spandrels Strong Piers (WSSP) behavior. The model keeps explicitly into account the various parameters that determine the structural response, as the masonry mechanical properties ($G_{X,i}$ and $\tau_{k,X}$), the interstory height (h_i), the resistant area ratio

($\alpha_{X,i}$), the interstory drift limits ($\Delta_{S,LS4}$, $\Delta_{F,LS4}$, $\Delta_{S,LS3}$, $\Delta_{F,LS3}$). These quantities were assumed as random variables and their values, together with the corrective factors above mentioned, are defined in detail in Lagomarsino and Cattari (2014).

PGA is assumed as *IM* and the median response spectrum shape is that of soil B – type 1, according to EC8 (CEN 2004). The epistemic uncertainty on the hazard curve was assumed $\beta_H=0.2$, while the uncertainty due to the response spectrum shape (β_D) was obtained, by considering the following ranges of characteristic values for the normalized acceleration response spectrum: $S_{a1,16}(T_c)=1.1$ and $S_{a1,84}(T_c)=0.9$.

The LS thresholds have been obtained by using a $D_{LS2} = 1.5\ D_Y$ and from proper drift limits.

Once defined the above-mentioned parameters, the fragility functions are obtained for each building class and for each number of stories; as in this case classes of height are considered (Low-rise, Medium-rise, and High-rise), fragility curves are obtained by a weighted average of the fragility curves for each number of stories. To this end, for Low-rise sub-class, it is assumed that 80% have two stories and only 20% are one-story buildings; for Medium-rise and High-rise buildings, a uniform distribution of height was considered.

Figure 4 shows, as an example, fragility curves of *DS2* (a) and *DS3* (b) for classes URM2-L and URM2-M. It is worth noting that the fragility curve obtained from the weighted average is not a lognormal cumulative function, but the parameters of a lognormal can be evaluated by least squares regression. At least for this class, it is worth noting that the building height has a bigger influence on DS2 than on DS3.

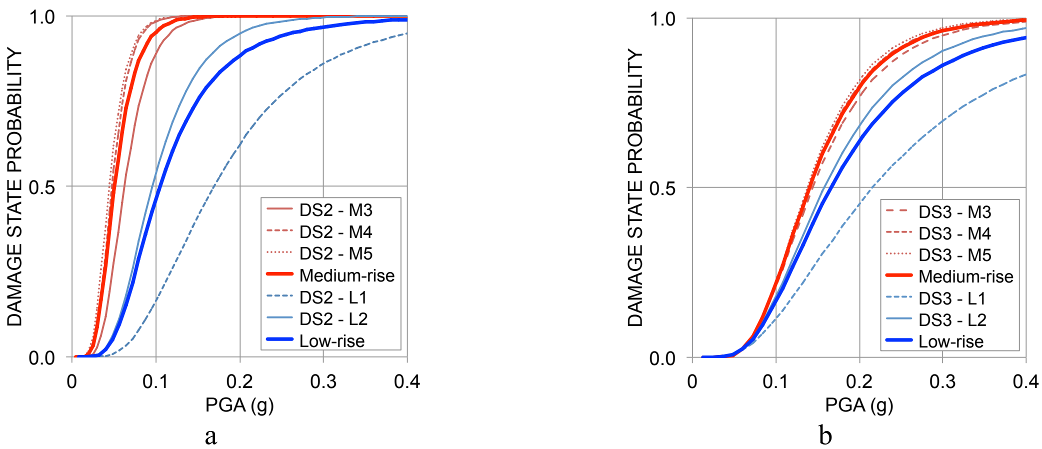

Figure 4. Fragility curves of DS2 and DS3 for URM2-L and URM2-M classes: both the curves for single number of stories and the weighted average for height classes are shown (Lagomarsino and Cattari, 2014)

In Table 2 the two parameters of the fragility functions ($IM_{LS} = PGA_{LS}$ and β_{LS}) for each damage state and the 10 building classes considered in this section are listed. It is worth noting that the dispersion β_{LS} is, on average, equal to 0.51; only in very few cases it is lower than 0.42 or greater than 0.6. The values of β_{LS} are similar but a little bitt lower than those obtained from the macroseismic vulnerability method for which they vary from 0.46 to 0.49 adopting the correlation law proposed in Faccioli and Cauzzi (2006), namely as (a) in Figure 5, and from 0.6 to 0.62 adopting that proposed in Murphy and O'Brien (1977), namely as (b) in Figure 5. This is correct, because greater uncertainties are implicitly included in the macroseismic approach, which has been derived by expert elicitation (derivation from the EMS98) and requires the establishment of a correlation between Intensity and *PGA*.

In order to validate the reliability of the obtained fragility functions from analytical procedure, a comparison with those produced by the macroseismic vulnerability method (Giovinazzi and Lagomarsino, 2006) is presented. The macroseismic model is based on the definition of the vulnerability index *V* and the ductility index *Q* as parameters representative of the seismic behavior of the vulnerability classes. For each class a plausible range of values of *V* is defined (for this reason in the Figure 5, two values of *V* are assumed for each class), instead, the ductility index is assumed equal to 3.

To this end, Figure 5 compares, for all the examined building classes, the values of PGA_{LS} obtained by the analytical model with plausible ranges estimated by the macroseismic vulnerability model (considering the corresponding wider building class of EMS98, taking into account only of the building height category). The curves show a good agreement, remaining almost always in the range created by the different correlations, used to convert I in PGA. For what concern the analytical approaches, it is obvious that it is more accurate, in fact it takes into account the presence of irregularity, that create a lower curve highlighting a bigger vulnerability. In general a more detailed approach involves in more conservative results.

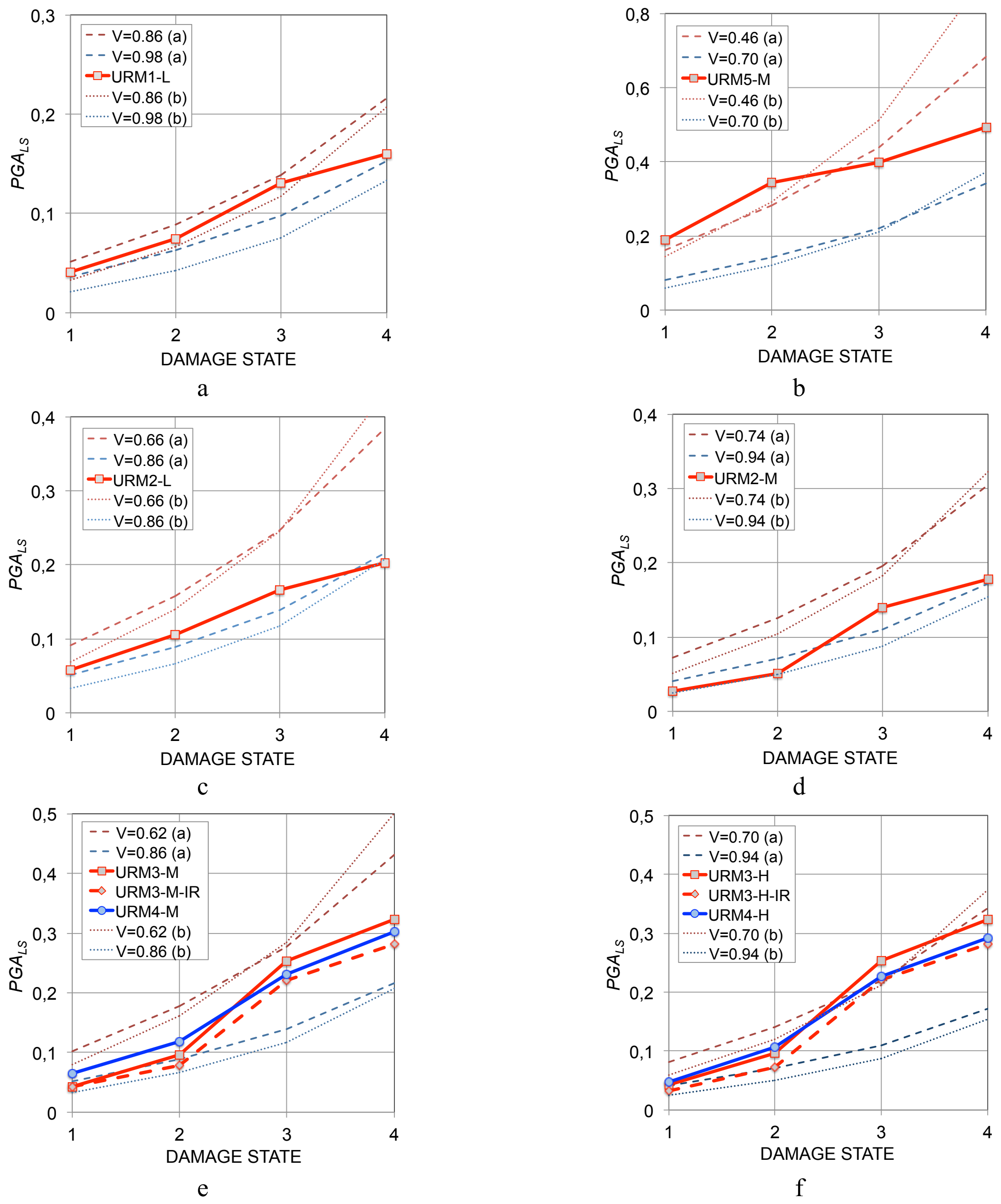

Figure 5. Comparison between PGA_{LS} values obtained from the simplified mechanical model and the macroseismic method for the 10 building classes considered (Lagomarsino and Cattari, 2014)

Table 2. Values of PGA_{LS} and β_{LS} of the fragility functions for the building classes examined

Class	DS1		DS2		DS3		DS4	
	PGA_{LS}	β_{LS}	PGA_{LS}	β_{LS}	PGA_{LS}	β_{LS}	PGA_{LS}	β_{LS}
URM1-L	0.041	0.65	0.074	0.61	0.131	0.53	0.160	0.47
URM2-L	0.057	0.52	0.105	0.53	0.166	0.54	0.203	0.505
URM2-M	0.026	0.475	0.051	0.40	0.140	0.43	0.178	0.41
URM3-M	0.057	0.50	0.104	0.49	0.253	0.56	0.325	0.61
URM3-H	0.043	0.43	0.096	0.38	0.253	0.54	0.323	0.60
URM3-M-IR	0.043	0.48	0.079	0.44	0.220	0.50	0.282	0.54
URM3-H-IR	0.032	0.42	0.072	0.37	0.221	0.49	0.281	0.53
URM4-M	0.065	0.51	0.118	0.52	0.231	0.55	0.302	0.59
URM4-H	0.048	0.43	0.107	0.39	0.226	0.51	0.292	0.56
URM5-M	0.189	0.54	0.343	0.58	0.399	0.79	0.492	0.68

CONCLUSIONS

This study presents two procedures to derive fragility curves from different vulnerability methods and their example application. For the first, it is important to note that each component of the dispersion coefficient can be evaluated analytically, differently from other studies that numerically estimate just one component (due to the demand) of the overall uncertainty. In the paper also the definition of a suitable taxonomy and the criteria to be adopted for a consequent classification of the built environment are discussed. These steps are essential in the settlement of the exposure model.

Considering the results, the different approaches show a good agreement. Therefore, the paper proposes the combined use of the two approaches:

- the analytical model, that has the advantage to keep explicitly into account the various parameters that determine the structural response of each vulnerability class. It allows a more detailed classification of the assets present in hazard zones that are thereby subjected to potential losses;
- the expert elicitation based models, that are obtained from observed damage and directly correlated to the actual seismic behavior of buildings.

Thus, the latter becomes the fundamental support to make reliable the results from the analytical method.

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