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LOSS ASSESSMENT OF UNREINFORCED MASONRY BUILDINGS

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Abstract

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. In particular, the outcome of a seismic risk analysis is the mean annual rate of specific consequences, for example economic loss, which is obtained by the probabilistic convolution of the three components: hazard, vulnerability and exposure. These types of analyses are increasingly directed to the evaluation of the socio-economic consequences of the earthquake, which represent a critical aspect that requires more research than others components of risk, since in the past most of attention has been addressed to the hazard and vulnerability. For this reason, the paper provides a contribution for the assessment of economic losses in masonry buildings, that implies the definition of a methodology on how to pass from structural seismic response to a response in terms of losses, following a component-based approach from which the direct loss is calculated by summing the losses over all damageable components in the building (structural, non structural and contents). The losses can be defined according to different approaches: passing directly from the intensity measure to repair cost or passing through firstly, the fragility curves, that relates the intensity measure and the engineering demand parameter, and, secondly, the engineering demand parameter and loss functions to estimate the repair costs. The first derives from the definition of a cost function that takes into account the progressing of damage in each structural element and the possible consequence on the reparability of related parts in the building. The procedure proposed is applicable for the detailed assessment of a specific building. The different inherent uncertainties, both aleatory and epistemic, which are involved in the seismic risk assessment (hazard, seismic demand, structural capacity and loss evaluation) are considered in the procedure. It is applied to a three-story unreinforced masonry case study analyzed with two configurations of constructive details aimed to simulate and compare two different in-plane collapse mechanisms that can generate a different losses assessment. These buildings are modeled by the equivalent frame approach (piers and spandrel beams) and analyzed by nonlinear static (pushover) and dynamic analysis following the Multiple Stripe Analysis.

Keywords: Seismic risk, Loss evaluation, Unreinforced masonry building

1. Introduction

There is a number of significant decision variables that need to be considered in the assessment of the seismic risk like as: the physical damage to buildings and other facilities; the casualties; the potential economic losses due to the direct cost of damage and to indirect economic impacts; the loss of function in lifelines and critical facilities; and, also but not less relevant, social, organizational and institutional impacts. In this framework, the paper aims to provide a contribution for the assessment of economic losses due to direct cost of damage in masonry buildings, for which the lack of literature is particularly noticeable. The proposed procedure complies with the aims and general framework of [1] and it is aligned to the PEER (Pacific Earthquake Engineering Research) PBEE (Performance-Based Earthquake Engineering) procedure [2], based on the following integral:

$$\lambda_{DV} = \iiint p(DV \mid DM)\, p(DM \mid EDP)\, p(EDP \mid IM)\, \lambda(IM)\, dIM\, dEDP\, dDM \tag{1}$$

As known, it is an example of total probability theorem that allows the disaggregation of the assessment problem into the four basic elements of hazard, structural, damage and loss (decision) analysis, by the introduction of the intermediate variables *DM* (Damage Measure), *EDP* (Engineering Demand Parameter) and *IM* (Intensity Measure of the hazard). The Eq. 1 implies the computation of : λ_{DV} the mean annual occurrence of a certain decision variable; *DV*, relative to a particular building, or class of buildings, and site, characterized by a specific hazard curve, $\lambda(IM)$. Furthermore, the general term $p(x|y)$ represents the probability density of x given y. The outcome λ_{DV} of the PEER PBEE methodology represents only one metric of performance, yet the seismic performance can consider numerous sources of loss expressed in a variety of metrics. These metrics can be annualized, such as Expected Annual Loss (*EAL*):

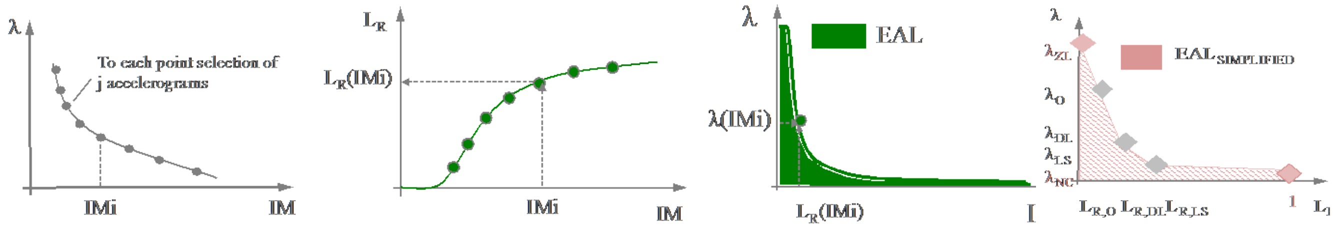
$$EAL = \int_{DV} f_{DV}(DV)\, dDV \tag{2}$$

In this framework, the masonry building-specific methodology presented herein is found on the losses to the individual components of a single building, according to the Component-Based approach [3]. The components are the parts, structural and non-structural, that all together comprise a building. According to this approach the direct loss is calculated by summing the losses over all damageable components in the building, that is the EAL_j of each component, that derives from the repair and replacement costs of them damaged during seismic events, properly weighed:

$$EAL = \sum_{j=1}^{n} EAL_j \cdot \alpha_j \tag{3}$$

where α_j is the economic weight of each component in a masonry building. It is worth noting that, from a in-depth analysis specifically addressed to masonry buildings (as illustrated in [4]) emerged that the repair costs of non-structural elements are not significant in this case, because, unlike the reinforced concrete building, they have a marginal impact since substantially almost all walls are structural.

The methodology based on the execution of detailed nonlinear dynamic analyses depends on three main steps: i) the definition of the hazard in terms of seismic hazard curve, with *i* values of *IMs* (Fig. 1.a) and sets of accelerograms (*j*); ii) the construction of the vulnerability curve (aimed to establish the relationship between repair costs - L_R - and intensity measures levels, Fig. 1.b); iii) the performance calculation, that entails the determination of the probable loss distributions and the computation of the expected annual loss (Fig. 1.c). Steps i) and iii) may be faced through the structural analysis of the building both in a rigorous and simplified way. The latter –conceived for being a practice-oriented procedure - passes through the identification of the limit states *LSs* and allows construing also a simplified loss curve, correlating the mean annual frequency of excedeence of each *LSs* and its economic losses ($L_{R,LS}$), as in Fig. 1.d, where a new *LS* is introduced, the Zero Loss limit state (*ZL*), as explained in [4, 5].



a. Hazard curve with i points   b. Vulnerability Curve   c – d. Loss Curve and EAL

Fig. 1 - Steps of the loss estimation procedure developed for masonry buildings in seismic area

Finally, the collapse mechanisms observed in a masonry structure can be traced back to two groups: the global response activation, with prevailing in-plane damage modes, and out-of-plane mechanisms, mostly occurred only on local portions of the structure. The methodology herein focuses only to the global response, by considering a box-type behavior.

In the following, the §2 describes the proposed procedure for assessing the earthquake losses of a single structure at a given site, while the §3 illustrates its application to two prototype buildings.

2. Loss estimation procedure for masonry buildings

The loss assessment procedure is firstly based on the selection of a representative *DV* that measures the seismic performance of the facility in terms of losses and consequences. In the proposal, the *EAL* is chosen as parameter to express the seismic risk: it represents the likely loss for any given year, seen as fraction of the overall value of the building. The internal parameters which the *EAL* is based on, that can include loss due to repair costs (L_R – loss ratio, defined as the percentage of the replacement cost) and collapse, constitute the *DVs*. The loss estimation, that implies the computation of the *EAL* and L_R, comes after a vulnerability analysis that, in the procedure proposed, may be established as follows:

- passing directly from the intensity measure to repair cost (IM-L_R), Fig. 1.b. This is based on the definition of a cost function directly dependent on a specific *EDP* as a function of the given examined component. For example in the case of vertical structural elements the assessment is based on an analytical cost function dependent on the drift of piers and spandrels, instead, for the floors, the variable that describes the replacement cost is the angular deformation;
- passing through the definition of incremental dynamic analysis (IDA*)* curves, that may be based on a rigorous incremental analysis or on similar techniques (i.e. the Multiple Stripe Analysis –MSA) by properly post-processing the results. Such second approach implies: a *IM-EDP* relation that is a description of the structural seismic response or “demand” versus the *IM*; the estimation of the seismic capacity of the structure; the determination of a probabilistic characterization of the variability of capacity and demand, and therefore the fragility function for the structural model considered [6]. After the definition of the fragility, the consequence functions (Damage level, *DL* - L_R) are introduced to estimate the repair costs.

For some components, for example the non-structural ones, both approaches can be applied, since for them it is possible to define an *EDP* representative of their structural behavior directly related to their cost. Vice versa for other components, that is the structural ones, only the first approach is feasible, because, it is limitative to consider their response in terms of loss described by a single *EDP*, in fact the losses are the sum of the costs of individual elements but also the result of the interaction of the response of the various structural elements.

From the first approach, the vulnerability curve (IM - L_R) on structural elements is obtained immediately. Introducing the hazard curve that reports the Mean Annual Frequency (*MAF*), λ_{IM}, of the relevant *IM* (with which the nonlinear dynamic analysis, NLDA, are performed), the loss curve ($\lambda_{IM}(im)$ - L_R) can be construed (Fig. 1.c). By taking the area under the loss curve the *EAL* is computed through the Eq. (2).

From the second approach, the IDA curve (*IM* - *EDP*) is determined. Introducing the occurrence of different damage levels or limit states in the IDA and the SAC/FEMA approach [7, 8], the mean annual frequency of the different *LSs*, λ_{LS}, are evaluated. The introduction of the *LSs* and the SAC/FEMA formulation is also essential for the evaluation of the simplified loss curve (as introduced in Fig. 1.d), that correlates the λ_{LS} and its economic losses ($L_{R,LS}$) that are chosen from reliable consequence functions.

In particular the components to be considered for a masonry buildings are listed in Table 1. For each one the following parameters are reported: the corresponding *EDP* assumed as representative of their seismic response; the unit of measure for the costs, in terms of volume of piers or partitions (obtained from the area A_p or $A_{pa,l}$ multiplied by the interstory height h_l), area of diaphragms or ceilings, and number of spandrels, respectively; the cost C_x for each element (x) per square meters or per unit; the total loss of each components (L); the percentage of the loss of each component than the overall losses. The values C_x are defined in [4] and they correspond to the maximum repair value, equivalent to the reconstruction value. Instead the last two columns of the Table 1 are the results that can be obtained from the procedure.

Table 1 - Inventory of the components for a masonry building

N° ID	Components	EDPs	Unit of measures	Cost	Loss	%
1. Structural	Piers	θ_p	A_p (p=1,N_p) h_l (l=1,N_l)	C_p = 350 €/m^2	L_w	L_w/L
	Spandrels	θ_s	N_s	C_s = 200 €/m^2		
	Diaphgrams	γ_s	A_d (d=1,N_d)	C_d = 70 €/m^2	L_d	L_d/L
2. Non-structural	Partitions	θ_l	$A_{pa,l}$ h_l (l=1,N_l)	C_{pa} = 650 €/m^2	L_{pa}	L_{pa}/L
	Ceilings	PFA	$A_{c,l}$ (l=1,N_{l+1})	C_c = 32 €/m^2	L_c	L_c/L
3. Contents		PFA	-	C_{co}	L_{co}	L_{co}/L

In the methodology proposed, the structural model of reference is the equivalent frame model according to which the structural wall is discretized in piers and spandrels, where the nonlinear response is concentrated, connected by rigid area (nodes). Piers are the main vertical resistant elements carrying both vertical and lateral loads; spandrel elements, which are intended to be those parts of walls between two vertically aligned openings, are secondary horizontal elements (for what concerns vertical loads), which couples the response of adjacent piers in the case of lateral loads. For this reason in the Table 1, the structural components are divided in piers and spandrels. If other strategies of modeling are adopted the components for the structural part may change. Despite this, it is important to note that the methodology herein proposed maintain a certain generality and could be properly adapted to other modeling strategies, as the finite element approach, through simple ex-post processing operations. In fact, apart the aims of equivalent frame approach, the classification of masonry panels in piers and spandrels is quite common in the literature and also the codes at national and international scales [9, 10] refer to such hierarchy in the structural masonry elements.

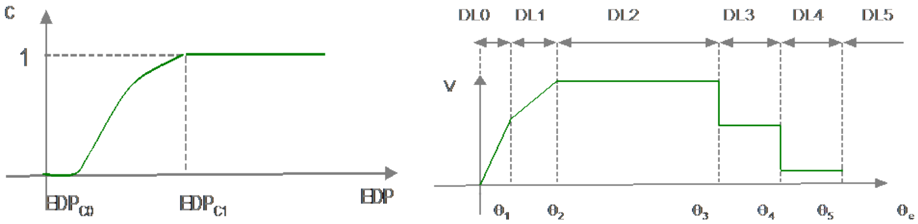
2.1 Loss model for the structural elements

2.1.1 The analytical cost function

The loss assessment procedure of structural elements is based on the definition of a cost function (Fig. 2.a), for walls and floors. The cost function of the walls considers the diffusion of damage in piers and spandrels. Generally, the conventional repair cost function of a single element is defined as: *c(EDP)*, where the *EDP* is representative of the specific elements. This function can vary with a linear or nonlinear trend between the zero value, for *EDP* equal to a lower threshold EDP_{C0} (which can be zero), and the value one for a *EDP* equal or greater than a higher threshold EDP_{C1}. In case of the three structural elements, the representative *EDPs* are: the drift for the elements (θ_e), piers (θ_p) and spandrels (θ_s), and the horizontal drift for the diaphragms (γ_d). For the purpose of the cost function, the threshold EDP_{C0} is the drift for which some damage occurs (θ_1 referred to the Damage Level 1 of the element, Fig. 2.b), while the threshold EDP_{C1}

defines the state beyond which it is not convenient repair the damage (e.g. θ_3 referred to the Damage Level 3 of the element, Fig. 2.b). A simplified approach, adopted in the case study following described (§3), is to assume a linear cost function for the piers and spandrels, Eq. (4), and diaphragms Eq. (5):

$$c_e = \begin{cases} 0 & \text{If } 0 \leq \theta_e < \theta_1 \\ \dfrac{\theta_e}{\theta_{C1,e}} & \text{If } \theta_1 \leq \theta_e < \theta_3 \\ 1 & \text{If } \theta_e > \theta_3 \end{cases} \quad (4) \qquad c_d = \begin{cases} 0 & \text{If } 0 \leq \gamma_d < \gamma_1 \\ \dfrac{\gamma_d}{\gamma_{C1,d}} & \text{If } \gamma_1 \leq \gamma_d < \gamma_3 \\ 1 & \text{If } \gamma_e > \gamma_3 \end{cases} \quad (5)$$



a. b.

Fig. 2 - Sketches of: a) Repair Cost Function c(EDP); b) Constitutive models of the elements of the reference model Tremuri [11]

The normalized cost function can be detailed for piers and spandrels, taking into account the actual costs of strengthening projects for masonry buildings [4].

2.1.2 The loss ratio evaluation

From the above-mentioned cost function *c(EDP)*, two variables representative of the total repair cost of the walls L_w (Fig. 3.a) and diaphragms L_d are introduced, where the sums are extended over the all piers ($N_p=\Sigma N_{plw}$), spandrels (N_s) and diaphragms ($N_d=\Sigma N_{dl}$):

$$L_w = \sum_{l=1}^{N_l}\sum_{w=1}^{N_w}\sum_{p=1}^{N_{Plw}} A_p h_l C_p c_p(\theta_p) + \sum_{s=1}^{N_s} C_s c_s(\theta_s) \quad (6)$$

$$L_d = \sum_{l=1}^{N_l}\sum_{d=1}^{N_{dl}} A_d C_d c_d(\gamma_d) \quad (7)$$

It is important to note that, the total repair cost of the structural elements is also influenced by the global response of the building: for example, when the soft-story mechanism occurs the total repair cost of the walls L_w increases. In fact, when in a wall w^*, at the generic level l^*, the interstorey drift of the walls ($\theta_{w,l}$) exceeds a given threshold θ_{l*}, the total repair cost functions of the walls and diaphragms have a sudden growth (Fig. 3.b). It is due to the fact that all piers and diaphragms located in that wall in the higher levels are considered to be rebuilt, that is they assume the value of the cost function equal to 1. So the increase is estimated according to the following formula:

$$\Delta L_{w*} = \sum_{l=l^*+1}^{N_l}\sum_{w=w^*}^{w^*}\sum_{p=1}^{N_{Plw}} A_p h_l C_p c_p(\theta_p) \quad (8)$$

$$\Delta L_{d*} = \sum_{l=l^*+1}^{N_l}\sum_{d=1}^{N_{dl^*}} A_d C_d c_d(\gamma_d) \quad (9)$$

Where N_{dl*} is the number of diaphragms supported by the walls w^* subjected to the soft-story mechanism.

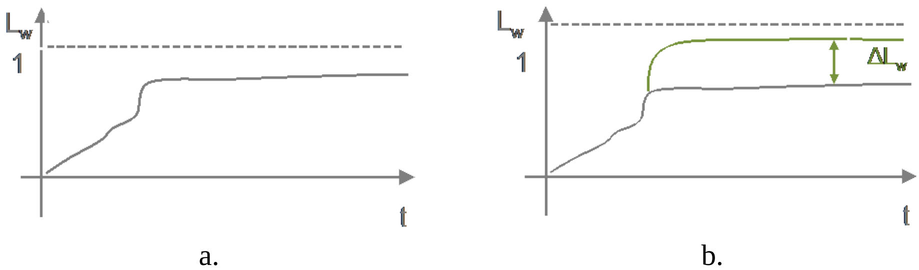


Fig. 3 - a) Sketch of the trend of the losses of the walls for a generic analysis; b) increment of the loss due to the occurrence of the soft-storey mechanism

The maximum repair costs for walls and diaphragms, associated to the complete damage of all elements, are given by:

$$L_{REPAIR,w} = \sum_{l=1}^{N_l} \sum_{w=1}^{N_w} \sum_{p=1}^{N_{Plw}} A_p h_l C_p + \sum_{s=1}^{N_S} C_s \tag{10}$$

$$L_{REPAIR,w} = \sum_{l=1}^{N_l} \sum_{w=1}^{N_w} \sum_{p=1}^{N_{Plw}} A_p h_l C_p + \sum_{s=1}^{N_S} C_s \tag{11}$$

The repair loss ratio can be obtained by normalizing Eq. (6) and Eq. (7) through the maximum repair cost, Eq. (10) and Eq. (11); under the hypothesis that C_p, C_s and C_d are constant all over the building, the following formulas come out:

$$l_w = \alpha_p \left[\frac{\sum_{l=1}^{N_l} \sum_{w=1}^{N_w} \sum_{p=1}^{N_{plw}} A_p h_l c_p(\theta_p)}{\sum_{l=1}^{N_l} \sum_{w=1}^{N_w} \sum_{p=1}^{N_{plw}} A_p h_l} \right] + (1-\alpha_P)\left[\frac{1}{N_S} \sum_{s=1}^{N_S} c_s(\theta_s) \right] + \Delta l_{w^*} \tag{12}$$

$$l_d = \frac{\sum_{l=1}^{N_l} \sum_{d=1}^{N_d} A_d c_d(\gamma_d)}{\sum_{l=1}^{N_l} \sum_{d=1}^{N_d} A_d} + \Delta l_{dl^*} \tag{13}$$

$$\Delta l_{w^*} = \alpha_p \frac{\sum_{l=l^*+1}^{N_l} \sum_{w=w^*}^{w^*} \sum_{p=1}^{N_{Plw}} A_p h_l c_p(\theta_p)}{\sum_{l=1}^{N_l} \sum_{w=1}^{N_w} \sum_{p=1}^{N_{Plw}} A_p h_l} \tag{14}$$

$$\Delta l_{d^*} = \frac{\sum_{l=l^*+1}^{N_l} \sum_{d=1}^{N_{dl^*}} A_d c_d(\gamma_d)}{\sum_{l=1}^{N_l} \sum_{d=1}^{N_{dl}} A_d} \tag{15}$$

The L_R of whole structural elements of the building (piers, spandrels and diaphragms) is obtained by normalizing the total repair costs of walls and diaphragms (L_w+L_d) by the total replacement cost ($L_{REBUILD}$); the latter is given by the rebuilding cost if this is lower than the cost of repair of whole elements ($L_{REPAIR,w}$+$L_{REPAIR,d}$). Also in other PEER documents [2], losses are expressed in terms of a Mean Damage Factor (*MDF*) that represents the percentage of the replacement cost of the buildings [12]. The L_R is given by:

$$L_R = \frac{l_w L_{REPAIR,w} + l_d L_{REPAIR,d}}{L_{REBUILD}} = \chi\left[\alpha_w l_w + (1-\alpha_w) l_d\right] \tag{16}$$

$$\chi = \frac{L_{REPAIR,w} + L_{REPAIR,d}}{L_{REBUILD}} \tag{17}$$

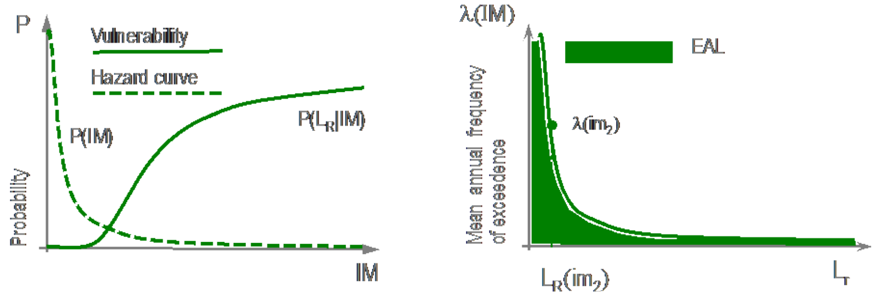
$$\alpha_w = \frac{L_{REPAIR,w}}{L_{REPAIR,w} + L_{REPAIR,d}} \tag{18}$$

As abovementioned, through the execution of nonlinear dynamic analyses, for each record of a given intensity measure, the loss ratio relative to the structural elements is defined in order to obtain the vulnerability curve (Fig. 4.a). This complies to perform an intensity-based and scenario-based assessment [1].

From the data of the vulnerability curve, if the number of intensities cover the entire hazard curve (Fig. 4.a) for the site it is possible to draw the loss curve (Fig. 4.b) as a series of points: Mean Annual Frequency, $\lambda_{IM}(im)$ and Loss Ratio (L_R). The *EAL* corresponds to the area under the loss curve (Eq. (19), [13]). With the definition of the loss curve the time-based assessment is achieved:

$$EAL = \int_0^{\infty} L_R \left| dPL_R \right| \tag{19}$$

where $P(L_R)$ is the probability of loss ratio exceeding a specified value L_R.



a. b.

Fig. 4 - a) Convolution of hazard and vulnerability; b) Loss curve with the identification of the EAL

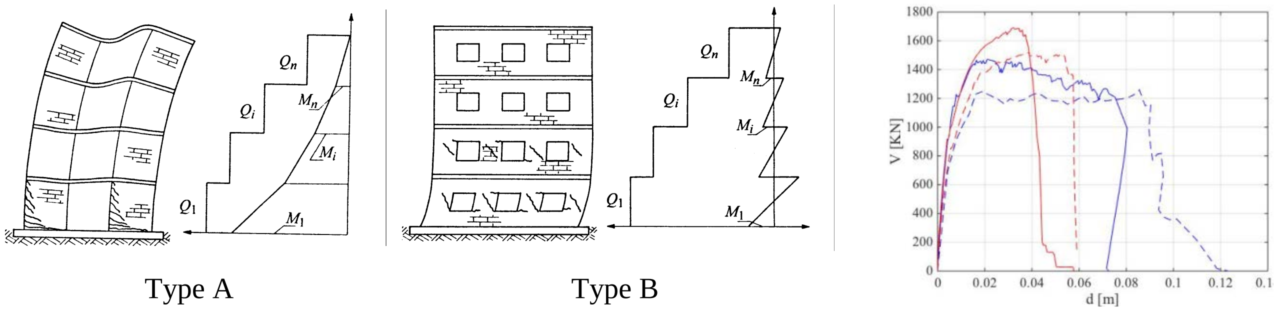
3. Application to a case study

In order to carry out the loss assessment at scale of a single unreinforced masonry (URM) building and to illustrate the effectiveness of the analytical procedure proposed in §2, a 3 storey ordinary building has been selected as case study. With reference to the structural systems, it is important to note that the URM buildings can have specific constructive details that can influence significantly the seismic behavior [14], the collapse mechanisms and, consequently, the direct losses. In this application, the case study examined has been analyzed with two configurations of constructive details conceived to simulate two different global collapse mechanisms. It is worth recalling that only the in-plane seismic response is herein considered.

The case study has a plan of 14 x 10 m and a height of 10.8 m. The structure is built in solid bricks and lime mortar. The thickness of the exterior walls varies from 48 cm in the first two storey to 36 cm in the last one, instead the interior walls have a constant thickness along the height and equal to 24 cm. The structure is isolated and the configuration plan is simple and regular; also the fronts have a regular windows disposition in the *x* direction and irregular in the *y* direction. As aforementioned two configurations of constructive details have been considered named as follows:

- Type A (Fig. 5.a), with wooden floors and roof, associated to spandrels coupled with tie-rods, located in each floor, in correspondence of the four exterior walls. Such configuration is representative of a very common type of existing masonry buildings and often produces a uniform collapse mechanism;
- Type B (Fig. 5.b), with rigid floors and roof, associated to spandrels coupled with reinforced concrete (RC) ring-beams. This model should cause a soft-storey collapse mechanism and is representative of existing buildings which suffered some strengthening interventions.

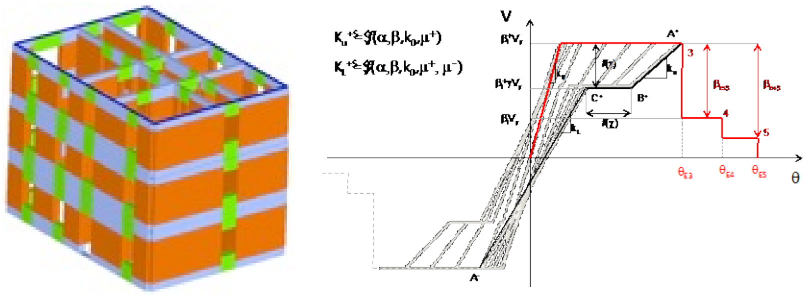
Both configurations, thanks to the presence of tie-rods and RC ring-beams, are not particularly vulnerable to the activation of out-of-plane mechanisms making licit the assumption of focusing only on the global in-plane response.



a. b.

Fig. 5 – a) Sketches of the different collapse mechanisms analyzed, adapted from [15]; b) Comparison of the response of the two configurations in terms of pushover curves

According to the Equivalent Frame (EF) modeling approach, the analyses discussed in the following sections have been performed by Tremuri software [16, 17] generating the structural model also through the support of the commercial version of the program [18]. Thus the complete 3D model is obtained (Fig. 6.a): i) by assembling 2D walls, by assuming the full coupling among the connected walls (hypothesis consistent for the original and strengthened state of the building) and condensing the degrees of freedom of two 2-dimensional nodes incident; ii) and by modeling floor as orthotropic membrane finite elements. Concerning the modeling of masonry panels, the piecewise linear constitutive laws (Fig. 6.b), recently developed and implemented in Tremuri program [11], have been adopted. They are based on a phenomenological approach with generalized force-deformation relationship consistent with those adopted in [19]. The nonlinear response is described until very severe damage levels (from 1 to 5) through progressing strength decay (β_{Ei}) in correspondence of assigned values of drift (θ_{Ei}); moreover a quite accurate hysteretic response is included as well, essential requisite to perform nonlinear dynamic analyses.



a) b)

Fig. 6 - a) EF model of the case study; b) Sketch of the idealization of masonry panels response according to the multilinear constitutive laws implemented in Tremuri [11]

Once defined the structural model, in order to perform the loss assessment, an inventory of the structural and non-structural members as well as the contents of the building was composed. In Table 1, the maximum costs of different components considered in the case study are reported. In the following, only the loss assessment of the structural elements is carried out.

3.1 Seismic demand

Choosing an appropriate intensity measure is an important step, since it can have significant effect on the scatter of data. Current best practice for the first mode dominated structures is to use the 5% damped spectral acceleration at the fundamental period of the structure, also in this work it has been chosen, considering a period of 0.3s. For the purpose of the evaluation the building has been located in L'Aquila (AQ). For this site, the seismic action has been characterized in terms of: the discrete hazard curve (that shows the annual frequency of excedeence of ground motions having different intensities) and a set of time histories of the seismic motion.

The first has been drawn starting from the values of the intensity measure $IM = S_a(T_1=0.3s)$ obtained by the median uniform hazard spectra (UHS) provided in the national code for nine values of the mean return periods, ranging from 30 to 2475 years [9]. The median discrete curve has been interpolated with a power-law curve (Eq. 20), or, equivalently, a straight line in log-log coordinates. For the purpose of the loss assessment three intensity measures were added to the nine: two return periods less than 30 years and one greater than 2475 years.

$$H(im) = k_0 \exp\left(-k_2 \ln^2 im - k_1 \ln im\right) \tag{20}$$

Where k_0, k_1 are positive real numbers, representing the intercept and the slope of the fitted line, instead k_2 is the hazard curvature. In this case, the abovementioned parameters that minimize the errors are: $k_0 = 0.108$, $k_1 = 1.749$ and $k_2 = 0.247$ for the Eq. (20).

For what concern, the time histories, 10 different records selected for each return period were provided [20]. They have been applied with 10 random directions, for the NLDA.

3.2 Uncertainties

The loss assessment procedure proposed is found on a fully probabilistic approach that requires understanding the inherent uncertainty involved in the risk analysis, incorporating those of both aleatory and epistemic nature. Herein only aleatory uncertainties have been considered as specified in the following, while the latter are usually treated with the logic tree technique. The uncertainties examined are: the seismic intensity at the site, governed by the hazard function, the record-to-record variability, described by a set of records, the material properties and the parameters of the constitutive laws of piers and spandrels. The effect of the aleatory uncertainties is quantified by associating each of the selected ground motions (10) for each return period a distinct realization of the random variables by extraction from their respective probability distributions.

3.3 Structural analysis

Nonlinear dynamic analyses have been executed by the Multiple Stripes Analysis (MSA) [21]. It consists of running a series of inelastic dynamic time-history analyses at various levels of excitation, over a suite of earthquake records (each with two orthogonal horizontal and the vertical component). For the analyses, 10 different structural models were established, characterized by parameters generated in accordance with the probability distributions of relevant parameters as described in [22]. In the dynamic analysis, each model was associated to each record selected for each return period selected and defined as illustrated in §3.2. Thus, the variability of results from the MSA comes from the randomness in the input motion but also the uncertainty in the aleatory parameters for the capacity. Then, from the NLDA with MSA approach, at each level of *IM* the following values are computed and analyzed statistically:

1 for the determination of the losses of nonstructural elements: the *EDPs* for the different components in terms of the mean interstory drift and *PFA*. The set of points ($IM=S_a(T_1)$, *EDP*) is then fitted with a lognormal distribution in order to compute the median, the fractile values of 16% and 84% and β □□□□□ standard deviation of the variable's n

2 for the determination of the losses of structural elements (Fig. 7): the losses of walls and diaphragms. The set of 10 ($IM=S_a(T_1)$, L_R) points is fitted with a beta distribution (computing the maximum likelihood estimates of the beta distribution parameters *a* and *b* from the data) in order to compute the median, the fractile values of 16% and 84%. With the median and the fractile values of the losses, the vulnerability curve can be defined. The Fig. 8 illustrates the median ones.

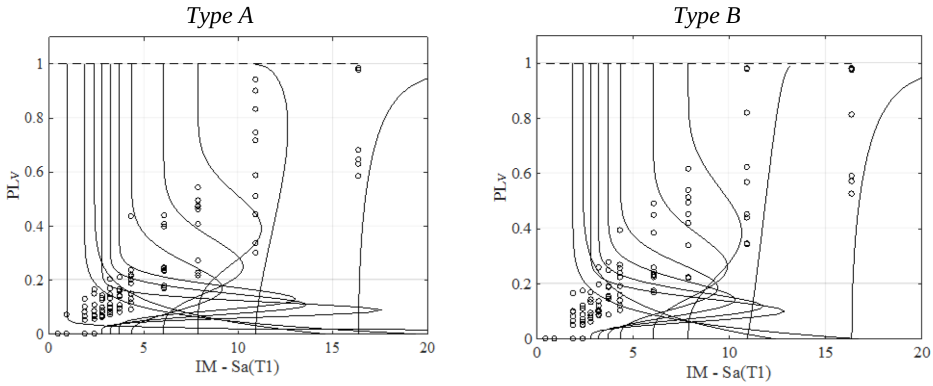


Fig. 7 - The set of 10 points *(IM, L_W)* fitted with a beta distribution

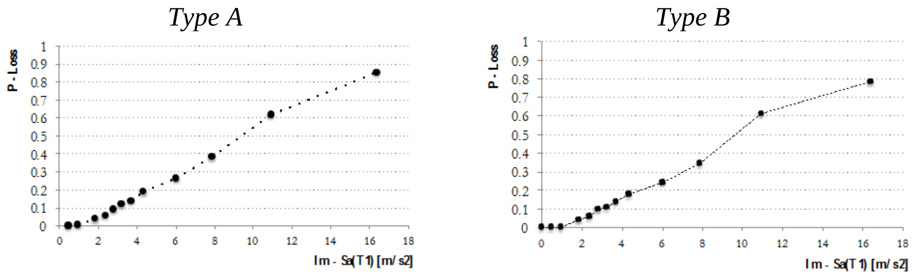


Fig. 8 - The median vulnerability curves

3.4 Loss Assessment

Once computed the variables introduced in §3.3, the estimation of the loss curve and *EAL* of the structural elements can be completed (Fig. 9).

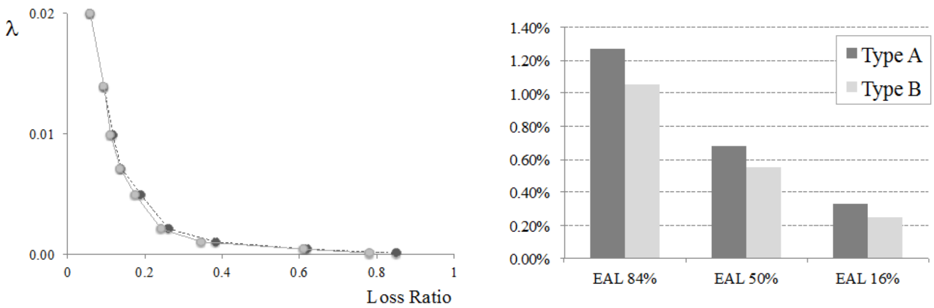


Fig. 9 - Median and fractile (16% and 84%) loss curves and *EAL* for the two cases study

In the cases study, the differences in the *EAL* is given by the more strength (although less ductile) of the Type B than the Type A (see Fig.5b). The final result of the two cases study is however comparable despite the initial difference in strength, this means that the Type A with the high intensities recovers the initial losses. The data of damage from past earthquake observations and also results from numerical seismic analysis would have suggested a more important difference between the damage and consequently losses of the cases study. This is not occurred because of the specific characteristic of this building and the properties assigned to the tie-rods and ring beams, that influence these different seismic responses.

In order to determine the global *EAL* of the building the $EAL_{NON\text{-}STRUCTURAL}$ must be summed to the $EAL_{STRUCTURAL}$ (Fig. 9), properly weighted. In the inventory of the case study, only the partitions are considered as non-structural elements, for which the economic weight is equal to 0.022, vice versa the structural weight is 0.681 (according to the inventory, [4], and the values of Table 1), therefore the $EAL_{STRUCTURAL}$ in the Fig. 9 is for the most part the global *EAL* of the building.

4. Conclusion

The main motivation of the work is the lack of reliable procedures to face the loss evaluation, above all for masonry structures, since in the past most attention has been addressed to hazard and vulnerability rather than losses. At the same time, there is a growing interest on this theme not only from the engineering community, but especially for insurers and reinsurers, government agencies and private businesses. For this reason, a loss estimation model for the computation of the expected economic losses for unreinforced masonry buildings in seismic area is presented. It combines the probabilistic seismic hazard of the area and the vulnerability models of the built environment, in order to estimate the extent of likely damage and the economic consequences through proper probabilistic indexes. The procedure proposed is an analytical methodology repeatable that could support in the future also extended parametric analyses on other prototype buildings, representative of different vulnerability classes of buildings for which it is interesting evaluate the loss curve and the *EAL*. The availability of these data is also essential to the loss evaluation at regional scale.

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