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SEISMIC COLLAPSE MECHANISMS ANALYSES AND MASONRY STRUCTURES TYPOLOGIES: A POSSIBLE CORRELATION

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SUMMARY: *In the framework of risk assessment at national and regional scale, the analysis of collapse mechanisms induced by seismic accelerations constitutes a useful tool to understand the behaviour of masonry structures and to plane mitigation strategies and rehabilitation interventions. In this perspective, the objective of the present study is the identification of the correlations among three factors: 1) structural-typologies which characterize Italian masonry buildings, 2) possible collapse mechanisms (in-plane and out-of-plane), 3) ground accelerations. The analyses developed concern a sample of 100,000 models representing the structural typologies of ordinary masonry buildings distributed on Italian territory. They have been derived through examination of structural characteristics (dimensions of structural elements, mechanical characteristics of material, typologies of horizontal structures and roofing, presence of vaults and/ or ties or ring beams, number of floors, etc.) collected by 'in situ' survey of about 250,000 buildings distributed along the Italian territory. For each model, the acceleration able to induce the first collapse mechanism has been calculated, adopting simplified limit state analyses. The results have been then elaborated to set, for different vulnerability classes (assigned combining typological- structural characteristics), the probability of occurrence of each collapse mechanism varying the peak ground acceleration (PGA) is analysed.*

KEYWORDS: *masonry structures, seismic vulnerability, collapse mechanisms.*

1 Introduction

In the last ten years, a large number of losses have been caused by earthquakes occurred in Italy (Abruzzo 2009, Emilia Romagna 2012, central Italy earthquake 2016).
Most of the casualties were caused by the collapse of the building structures, thus the strong interest to assess the seismic vulnerability of existing buildings to prepare seismic risk mitigation plans.
The aim of a vulnerability assessment is to obtain the probability of a specific damage level caused by a scenario earthquake to a given building type.
A good semantic definition of vulnerability is given by Sandi [1986]: "the seismic vulnerability of a building is its behaviour described by a cause-effect law, where the cause is the earthquake and the effect is the damage", however beside this a quantitative definition within the framework of the decision theory can be given: Vulnerability is the probability that

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an element at risk of a given typological class (i.e. A, B, C, …..) can accuse a level of damage (i.e. D1, D2, D3,…….) consequent to the action of a given level of hazard intensity (i.e. V, VI, VII,…….).
In order to perform vulnerability assessments of masonry buildings, several approaches, each one related to a different level of approximation, are available in the literature [Calvi et al., 2006]. For the reader's convenience, such strategies can be grouped in two categories:

- Observed vulnerability /Statistical approach.

The vulnerability is derived from the synthetic analysis of the formal and structural characteristics of the building.
A restricted number of building categories, called "vulnerability classes", are identified as a function of the typological and structural characteristics. Each class is then associated to an expected behaviour under seismic action, this behaviour is described by a vulnerability function that generally is calibrated by analyzing the damage observed during past events.
Applications of this method, using the Damage Probability Matrices (DPM), were originally proposed by Whitman et al. [1973], who analyzed the damages observed in more than1600 buildings after the 1971 San Fernando earthquake, by Braga et al [1982,1986] after the 1980 south-Italia earthquake and by Zuccaro et al [2000], Bernardini et al [2007a, 2007b].
The validity of this approach is reliable on a large number of buildings having characteristics to be included in specific vulnerability classes, obviously it is not reliable for single buildings. On the other hand it has the undeniable advantage of demanding both little information and rapid processing. Furthermore, it is derived from observations of the actual performance of assets in real earthquakes. For this reason it is useful for investigating a wide range of buildings (urban scale or wider). The research in this context aims to achieve a greater reliability of results while maintaining an acceptable agility of the investigation.

- Calculated Vulnerability/ Mechanical approach.

The vulnerability evaluation is the result of accurate computations using simplified limit state analysis on predefined categories of Structural Mechanics.
The damage evaluation is formulated on the basis of analytical calculations to determine the seismic response of the building, the stress and corresponding strain state are derived.
In this way, the problem of seismic vulnerability of masonry structures is developed in structural engineering terms. Vulnerability is computed as a direct function of construction characteristics, structural response to seismic actions and damage effects. Applications of this method can be seen in the work of Giuffrè [1991], Singhal and Kiremidjian [1996], Park and Ang [1985], Masi [2003], Rossetto and Elnashai [2005], Dumova-Jovanoska [2004], D'Ayala et al. [2015].
This approach provides assessments certainly more reliable on single buildings, but it requires detailed knowledge of technical features of the buildings and the development of time consuming structural calculations. Therefore it is difficult to implement at large scale.
Despite its robustness, the mechanical approach requires a detailed knowledge of structural features of the analyzed buildings and high computational efforts in computing the structural responses and, for these reasons, it is hardly implemented at a large scale.

In order to overcome the drawbacks of both these strategies, hybrid methodologies, aiming at combining the results of a simplified mechanical model with the vulnerability evaluations of a statistical approach, have been recently proposed. Their main philosophy consists in enriching a limited dataset, which is statistically analyzed, by the computation of simulated mechanical data or, alternatively, to introduce probabilistic corrections, derived by observed dataset, in a mechanical model, see, e.g., Cavalieri et al. [2017].

Following a hybrid approach, the procedure presented in this research aims to develop collapse probability distribution for a set of building classes suitable for the Italian structural typologies. In particular, the statistical data derived by the survey of about 250,000 buildings is adopted to randomly generate a virtual set of mechanical models where each instance is characterized by mechanical properties relevant to a typological class according to the vulnerability assessment method proposed by Zuccaro et al. [2012]. The structural models are analyzed by means of simplified limit state analysis procedures in order to evaluate their seismic response and to compute the failure probability curves, expressed in terms of seismic acceleration for each typological class.

The proposed algorithm is presented in Section 2, which also summarizes the definition of the statistical dataset, while the simplified structural analysis and the adopted collapse mechanisms are reported in Section 3. Numerical results are presented by discussing the failure probability curves relevant to each collapse mechanism, presented in Section 4, and by computing the vulnerability curves of each structural typology, shown in Section 5. The presented results are compared with alternative strategies, such as the mechanical approach proposed by Cattari et al., [2014] and the statistical procedure presented by Zuccaro et al. [2015]. A brief discussion on such comparisons and on the robustness of the proposed algorithm is reported in Section 6; finally, conclusions and future work are summarized in Section 7.

The approach adopted constitutes a preliminary study to understand the basic seismic behaviour of the ordinary masonry buildings. Further developments of the research will include additional improvements also in dynamic state, able to identify a more accurate evaluation of collapse accelerations [Boothby, 2001; De Jong, 2009], considering micromechanical modelling of failure (effects of deformations in the mortar joints, detailed properties of the material, irregularity in the panels, etc.) and more detailed sensitivity analyses on the observed data used and on their reliability.

2 Methodology

The procedure, aiming to determine the vulnerability curves as functions of the structural typology and of the seismic acceleration, involves different steps of both statistical and mechanical nature. As described in the introduction, the aim of the paper is to evaluate the correlations between the vulnerability factors of masonry buildings, based on their structural typologies, and the collapse mechanisms potentially triggered by the seismic action. The result has been obtained by a Monte Carlo simulation analysis, and can be described in the following steps:

i. *Statistical analysis of existing masonry buildings*. The analysis, pursued thanks to the PLINIVS Study Centre[1] Database, is based on the survey performed all along the

[1] PLINIVS Study Centre for Hydrogeological, Volcanic and Seismic Engineering. University of Naples, Italy. Centre of competence of Italian Civil Protection. www.plinivs.it

Italian peninsula. About 250,000 of the residential masonry buildings (ISTAT Census 2011) have been surveyed distributed on about 600 municipalities.
This analysis has allowed to investigate the geometrical and structural characteristics of Italian masonry buildings and identify the recurring combinations of these characteristics. The probability of combination between a particular characteristic (e.g. type of vertical structure) and other features (e.g. type of horizontal structure, presence of ties, etc.) have been then evaluated.

ii. *Iterative model generation*. An iterative procedure has been implemented by an ad-hoc software developed in order to generate virtual model of buildings (about 100,000). The program adopts a random assignment procedure of the structural characteristics whose probability distributions are known from previous step.

iii. *Seismic Vulnerability classification by "SAVE" method*[2]. The generated virtual buildings have been classified in vulnerability classes: A, B, C, D (EMS'98) on the basis of their typological characteristics. According to the assignment procedure based on the criteria defined in the "SAVE" project [Zuccaro et al. 2015], the vulnerability level of each building is evaluated considering the typological characteristic as vulnerability modifiers (Table 1), and giving to each of these a numerical weight calibrated using a wide database of seismic damage observed after the most important earthquakes occurred in Italy in the last forty years.

Table 1 -*Main typological characteristics identified on existing masonry buildings*

Typology of vertical structure	Solid bricks Regular stonework Tuff Hollow bricks Irregular stonework
Typology of horizontal structure (Intermediate slabs and roof)	Wooden beams Iron beams R.C. with hollow brick Vaults
Number of floors	1 to 5
Inter-storey height	3- 3,5- 4- 4,5- 5 m
Wall thickness	0,27 to 0,8 m
Wall length	4- 5- 6- 7 m
Percentage of openings	10-20-30-40%
Pushing roof	On/off variable
Horizontal embedding	On/off variable
Effectiveness of links	On/off variable

[2] "SAVE" - Updated Tools for the Seismic Vulnerability Evaluation of the Italian Real Estate and of Urban Systems. National Research project funded by Italian Government

iv. *Collapse Mechanisms calculation*. For each virtual building the trigger acceleration (a_g) responsible of the relevant Collapse Mechanisms have been computed. The mechanisms considered are assumed with reference to the classification adopted in the MEDEA[3] methodology [Zuccaro and Papa, 2002].

v. *Vulnerability curves assessment*. Collecting the obtained results, for each typological class (A, B, C, D), vulnerability curves are built, expressing the collapse probability as a function of the ground acceleration (a_g).

Details concerning the assumptions of the mechanical model as well as the statistics of the computed probability distributions are reported in the following sections while the details concerning the dataset of existing buildings surveys are omitted for brevity.

3 Collapse mechanisms

3.1 Preliminary remarks

The simplified limit state analysis proposed by the MEDEA strategy consists in modelling the analyzed structure by means of macro elements and in assuming a set of the collapse mechanisms which have been observed as the most significant in masonry structures. In particular, for each of the adopted limit states, the simplified analysis computes the mass multiplier α, significant of the horizontal loads, for which the macro-element model attains the ultimate limit state relevant to each considered mechanism. The corresponding value a_g of the trigger seismic base acceleration is computed by the typical relationship (1) [CM, 2009-C8A.4.9].

$$a_g = \frac{\alpha \cdot q}{S} g \tag{1}$$

where:

α is the horizontal mass multiplier;
q is the ductility factor;
S is the subsoil factor, fixed equal to 1.25;
g is the gravity acceleration (9,81 m/s^2);

It is worth to emphasize that value of the collapse acceleration depends on the typological and structural properties assumed for the analysed virtual model. Such structural features can be considered as *typological vulnerability factors* since their combination has a great influence on the building capacity.

Moreover, the confidence of the computational results depends on the definition of the ultimate limit states assumed for the structural model since they must be significant of the physical behaviour of the analysed building. Obviously, a limited set of collapse mechanisms or inaccurate assumptions concerning the macro-elements failure, can seriously compromise the capability in estimating reliable values of the ultimate loads. At the same time, it is desirable that the physical characterization of such conditions is defined in terms of a limited

[3] "MEDEA" A multimedia and didactic handbook for seismic damage evaluation. A self-training multimedia tool for seismic damage evaluation.

set of parameters acknowledged by the professional community as significant of the considered structural behaviour.
For this reason, the presented procedure is based on collapse mechanisms and ultimate limit states which have proved to be effective and significant of the structural typologies detected in the statistical dataset. These conditions, outlined for convenience as in-plane and out-of-plane mechanisms, are widely employed in the literature and commonly accepted as robust and reliable and are reported below [Novelli and D'Ayala, 2012; Fortunato et al., 2014; Iannuzzo et al. 2017].

3.2 In-plane Collapse mechanisms

In-plane collapse mechanisms, according to Giuffrè definition [Giuffrè, 1991], can be considered as "second mode mechanisms". The associated damage (shear cracks) generally does not lead the structure to collapse, but it can facilitate contemporary out-of-plane mechanisms.
Virtual models generated by the iterative procedure are assumed to be composed by rectangular walls with maximum two openings each. The simplified structural analysis adopts rectangular macro elements, therefore, in the case of panels presenting one or two openings an automatic partition is performed in order to analyse windowless rectangular panels. Loads acting on the original wall are distributed on the derived panels proportionally to their stiffness.
In general, several in-plane collapses present a classical X-shaped crack pattern due to diagonal orientations of the tensile-compressive principal stresses. Such a behaviour is typical of shear or slip failure which are assumed as the first two collapse mechanisms of the presented procedure. Moreover, it is considered a further collapse behaviour, although not as frequent as the previous ones, consisting in the vertical buckling of the rectangular panel.
It is worth being emphasized that, whenever the original wall is composed by more than a single panel, each sub-element presents a peculiar value of the collapse mass multiplier α and, subsequently, of the trigger acceleration a_g. In such a case, the original wall is assumed to behave as a series system so that its collapse multiplier corresponds to the minimum α computed in all its sub-elements.
All the considered in-plane collapse mechanisms are analysed by considering a rectangular panel with thickness s, height h and width l_p subject to vertical compression N and to a shear action T at the lower and upper base. In conclusion, the in-plane collapse mechanisms considered in this study are:

- Shear crack,
- Failure by slip,
- Buckling failure.

Their limit conditions are reported below.

Shear crack

The shear crack collapse mechanism is analysed by the Turnsek and Cacovic model [Turnsek V. et al, 1971].The assumed limit state condition consists in the opening of a diagonal crack, as shown in Figure 1, depending on the attainment of the tensile limit value by the stress components orthogonal to the crack direction.

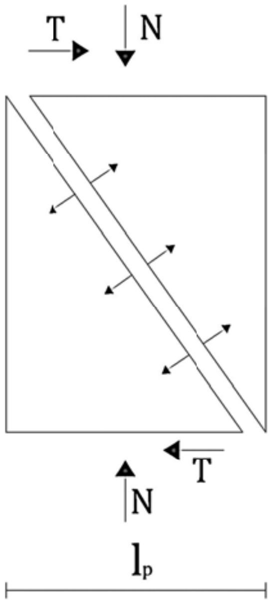

Figure 1 - *Shear crack mechanism*

Denoted by τ_k the characteristic value of the limit shear stress component and setting N equal to the normal compression strength, the shear strength T_u, i.e. the value of the horizontal shear corresponding to the attainment of the ultimate limit state, is computed as the equation (2):

$$T_u = (l_p \cdot s \cdot \tau_k) \cdot \sqrt{1 + \frac{N}{p \cdot (l_p \cdot s \cdot \tau_k)}} \tag{2}$$

where p is a coefficient accounting the distribution of the shear stress components over the panel. Usual values of p span between 1, in the case of thick walls, and 1.5, for thin panels.

Failure by slip

A similar limit state condition concerns the slip collapse mechanism shown in Figure 2, where the crisis of the panel is assumed to be activated by the attainment of the tensile limit of the stress component parallel to the crack.

In such a case, the slip shear strength T_u is computed by the relationship provided by the Italian structural code [NTC, 2008]:

$$T_u = l_p \cdot s \cdot f_{vk} \tag{3}$$

where f_{vk} is the characteristic value of the limit shear stress of the panel, computed by:

$$f_{vk} = f_{vk0} + 0{,}4 \cdot \sigma_n \tag{4}$$

with f_{vk0} denoting is the characteristic shear stress of the material and σ_n the normal compressive stress component acting among the panel.

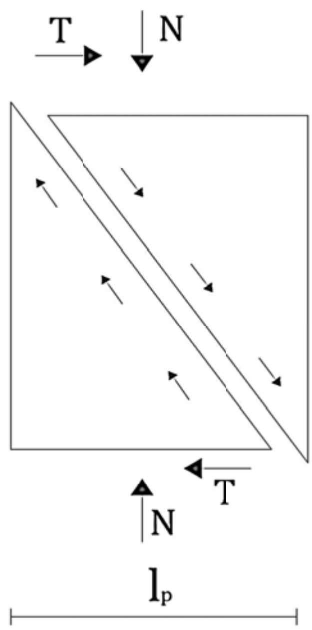

Figure 2 - *Failure by slip*

Buckling failure

The last in-plane collapse condition considered in this research models the crisis due to the combination of vertical compression and uniaxial bending of a rigid, rectangular panel.
Assuming a compressive-only behaviour of the material, the maximum values of the compressive stress, for which collapse is induced, are clustered at the extreme base region of the panel, as sketched in Figure 3. In fact, because of the rigid behaviour of the panel, bending induces a rigid rotation about the axis orthogonal to the middle plane.

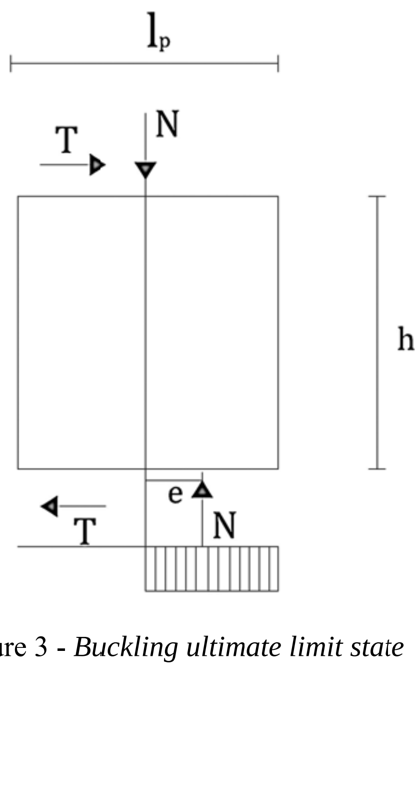

Figure 3 - *Buckling ultimate limit state*

The panel limit state is defined by the ultimate value of the bending moment M_u which is computed by a simplified equation provided by the Italian code [NTC, 2008]:

$$M_u = \left(l_p^2 \cdot s \cdot \frac{\sigma_n}{2}\right) \cdot \left(1 - \frac{\sigma_n}{0{,}85 \cdot f_d}\right) \tag{5}$$

where σ_n is the vertical compressive stress component of the panel and f_d is the characteristic value of the compressive limit stress of the masonry. Assuming the static constraint of the wall be fixed at the base and linked support at the top, by equilibrium considerations, the ultimate shear strength T_u of the panel is:

$$T_u = \frac{2 \cdot M_u}{h} \tag{6}$$

with h denoting the height of the panel.

3.3 Out-of-plane mechanisms

Out of plane collapse mechanisms consist in kinematically indeterminate displacements of one or more structural elements compromising the structural capacity of the model. Such behaviours can be induced by different phenomena, such as ineffective connections between contiguous walls, insufficient anchoring of the floors or out-of-plane horizontal loadings due to vaults, floors and roofs. In some cases, diagonal cracks due to in-plane mechanisms behave as cylindrical hinges although this last case usually occurs for high levels of overall damage.
Regardless of the nature of kinematic collapse, such mechanisms are particularly dangerous in absence of temporary retaining structures even if the incipient kinematic indeterminacy has not completely developed. In this respect, after the attainment of in-plane limit states, structural elements usually present a residual capacity due to their ductility so that the structural model is capable of achieve a sufficiently stable equilibrium condition. On the contrary, several cases of incipient out-of-plane collapse correspond to values of the potential energy which, although stationary, are not local minima; thus, even limited perturbations can trigger disastrous failure. For this reason, the analysis of such a class of collapse mechanisms is of outmost importance in estimating the structural vulnerability.
Because of the kinematic indeterminacy of the schemes modelling out-of-plane mechanisms, the estimation of the collapse multiplier α related to a structural panel must be performed by applying the virtual work theorem according to the Lagrange's procedure.
To this end, following the macro-element philosophy adopted in this analysis, the limit state conditions are computed by means of rectangular panels subject to a standardised set of loads. In particular, the generic panel is subject to *m* vertical loads P_{Si} due to the floorings; the horizontal load P_H due to the roof; the horizontal load Ti due to any tie-beams at top; and *r* static actions related to arches and vaults, characterized by means of their vertical F_{Vi} and horizontal F_{Hi} components where subscripts *i* are progressive indexes.
Each collapse mechanism is characterized by the presence of a cylindrical hinge; therefore, the panel is partitioned in *n* regions, whose self-weight is defined as W_i, which can rotate independently about the hinge's axis. The kinematic scheme determines the distances of the static actions with respect to the location of the hinge. Specifically, assumed the hinge as origin of a horizontal-vertical reference system, the centre of mass of the *i*-th sub-panel is

located at $[X_{Gi}, Y_{Gi}]$; the actions P_H and P_{Si} are applied at $[d_i, h_i]$, while the coordinates of actions F_{Hi} and F_{Vi} are $[d_{vi}, h_{vi}]$.
Analogously to the in-plane mechanisms, the simplified analysis aims to determine the collapse multiplier α of the horizontal loads which is computed by defining a rotational equilibrium relationship with respect to the hinge location:

$$\alpha\left(\sum_{n} W_i \cdot Y_{G_i} + \sum_{m} P_{S_i} \cdot h_i + \sum_{r} F_{V_i} \cdot h_{v_i}\right) + \sum_{r} F_{H_i} \cdot h_{v_i} + P_H \cdot h_i = \sum_{n} W_i \cdot X_{G_i} + \sum_{m} P_{Si} \cdot d_i + \sum_{s} F_{V_i} \cdot d_{v_i} + \sum_{n} T_i \cdot h_i \tag{7}$$

It is worth being emphasized that the left side of Equation (7) represents the momentum of the horizontal actions about the hinge axis. The presence of W_i, P_{Si} and F_{Vi}, formally vertical actions, is due to the fact that multiplier α is significant of the horizontal acceleration; therefore, the terms inside the brackets physically represent horizontal inertia forces. In this sense, the overturning moment at the left side of Equation (7), is in equilibrium with the resisting moment computed in the right side.
The trigger horizontal acceleration a_g is then computed by a relationship conceptually analogous to the one reported in Equation (1) which yields [CM, 2009- C8A.4.9]:

$$a_g = \frac{\alpha \cdot \left(\sum_{i=1}^{n} W_i + \sum_{i=1}^{m} P_i\right)}{M^*} \cdot \frac{q}{S} \tag{8}$$

where q is the ductility factor, S is the subsoil factor assumed to be equal to 1.25, g is the gravity acceleration and M^* is the participating mass. This last quantity is due to the presence of the non-uniform displacements distribution presented by the system of rigid sub-elements excited by the base acceleration and is computed by:

$$M^* = \frac{\left(\sum_{i=1}^{n} W_i \cdot d_{x,i} + \sum_{j=1}^{m} P_j \cdot d_{x,j}\right)^2}{g \cdot \left(\sum_{i=1}^{n} W_i \cdot d_{x,i}^2 + \sum_{j=1}^{m} P_j \cdot d_{x,j}^2\right)} \tag{9}$$

where $d_{x,i} d_{x,j}$ are the values of the virtual displacement computed at the application points of actions W_i and P_j, respectively.
The values of the weights and external actions and their locations are computed by the simplified structural analysis for each macro-element while the location of the kinematic hinge depends on the peculiar collapse mechanism considered in the analysis. In particular, each macro-element has been analyzed with respect to three out-of-plane failure schemes, namely:

- simple overturning,
- vertical bending,
- horizontal bending.

whose analyses are reported in the following subsections.

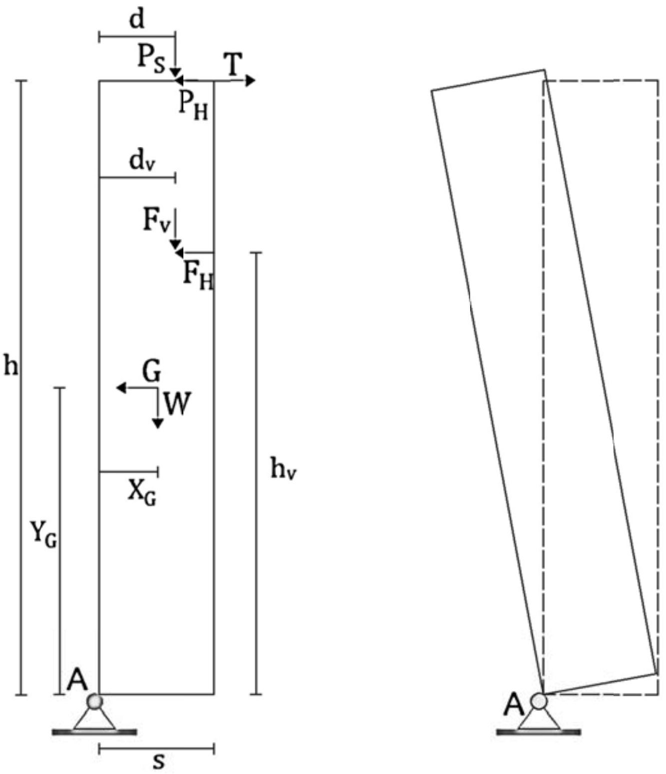

Figure 4 - *Simple overturning failure, vertical section of the wall*

Simple overturning

Simple overturning mechanism consists in a rigid rotation about one of the edges of the base section, as shown in Figure 4 where the wall is represented by its transverse vertical section and a counter-clockwise rotation is assumed. In such a case, the panel does not present any internal discontinuity; therefore, no partition is required.
Figure 4 also reports all the forces acting on the generic panel and contributing to the kinematic equilibrium; therefore, Equation (7) can be specialized as:

$$\alpha[W \cdot Y_G + F_v \cdot h_v + Ps \cdot h] = W \cdot X_G + F_v \cdot d_v + Ps \cdot d + T \cdot h - F_H \cdot h_v - P_H \cdot h \quad (10)$$

and the collapse multiplier turns out to be:

$$\alpha = \frac{W \cdot X_G + F_v \cdot d_v + Ps \cdot d + T \cdot h - F_H \cdot h_v - P_H \cdot h}{W \cdot Y_G + F_v \cdot h_v + Ps \cdot h} \quad (11)$$

representing the relationship proposed by Milano et al.(2008).

Vertical bending

The vertical bending mechanism (Figure 5) involves a vertical instability of the wall induced by the seismic inertial forces and the action of an intermediate floor. For this reason, the hinge is located in correspondence of an intermediate floor (point C) while the neighbouring

floors act as horizontal restraints (points A and B). Subsequently, the panel is partitioned in two sub-elements which can rotate about point C.
Moreover, Figure 5 shows a vertical transverse section of the considered model and reports all the assumed actions and their locations. The virtual work relationship is derived assuming a counter-clockwise rotation of the bottom sub-element.
As shown in Milano et al. [2008], Equation (7) can be specialized for this kinematic scheme; in particular, the resisting moment turns out to be:

$$\begin{aligned} E = {} & \frac{W_1}{2}s_1 + F_{v_1}d_{v_1} + \left(W_2 + P_{S2} + N + F_{v_2}\right)s_2 + \\ & + \frac{h_1}{h_2}\left(\frac{W_2}{2}s_2 + Ps_2 d_2 + Nd_n + F_{v_2}d_{v_2} - F_{H_2}h_{v_2}\right) + P_{S1}d_1 - F_{H_1}h_{v_1} + Th_p \end{aligned} \qquad (12)$$

so that the collapse multiplier is:

$$\alpha = \frac{E}{W_1 \cdot Y_{G_1} + F_{v_1} \cdot h_{v_1} + Ps_1 \cdot h_p + \left(W_2 \cdot Y_{G_2} + F_{v_2} \cdot h_{v_2}\right)\frac{h_1}{h_2}} \qquad (13)$$

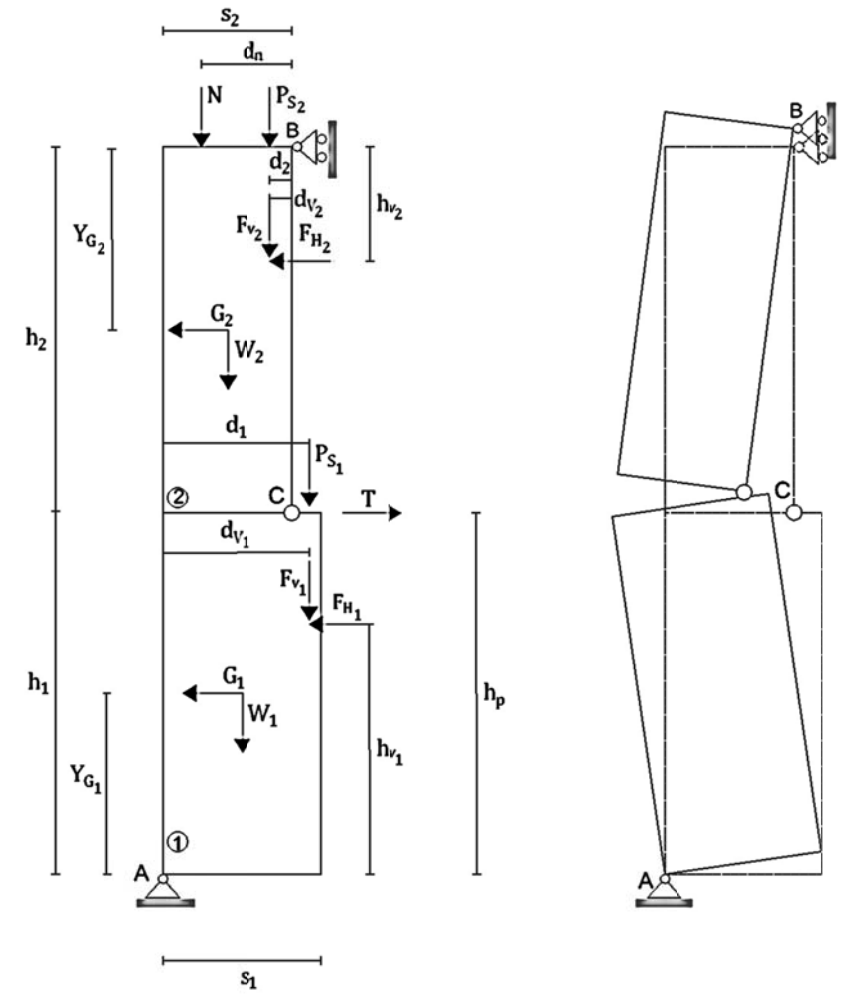

Figure 5 - *Vertical bending failure, vertical section of the wall*

Horizontal bending

The last out-of-plane mechanism considered in the performed analyses concerns horizontal bending of two adjacent panels. It is related to arch mechanisms induced, within the wall sections, by out-of-plane actions.

In such a case, three hinges, reported in Figure 6, are considered. Moreover, apart from the standard actions introduced before, a further contribution must be considered. It consists in a horizontal force accounting for architectural constraints, such as the action of steel ties, if present, and the friction between the panel and the floor slab.

The equilibrium condition of Equation (7), computed by the application of the rigid displacement and considering the distances reported in Figure 6, permits to determine the collapse multiplier which turns out to be:

$$\alpha = \frac{T \cdot s\left(1 + \frac{l_{p_1}}{l_{p_2}}\right) + \sum_i P_{H_{i1}} \cdot d_{H_{i1}} + \sum_i P_{H_{i2}} \cdot \frac{l_{p_1}}{l_{p_2}} \cdot d_{H_{i2}}}{W_1 \cdot X_{G_1} + W_2 \cdot \frac{l_{p_1}}{l_{p_2}} \cdot X_{G_2} + \sum_i P_{S_{i1}} \cdot d_{H_{i1}} + \sum_i P_{S_{i2}} \cdot \frac{l_{p_1}}{l_{p_2}} \cdot d_{H_{i2}}} \tag{14}$$

where T is the maximum resistance due to architectural constraints (see, eg., Milano et.al 2008).

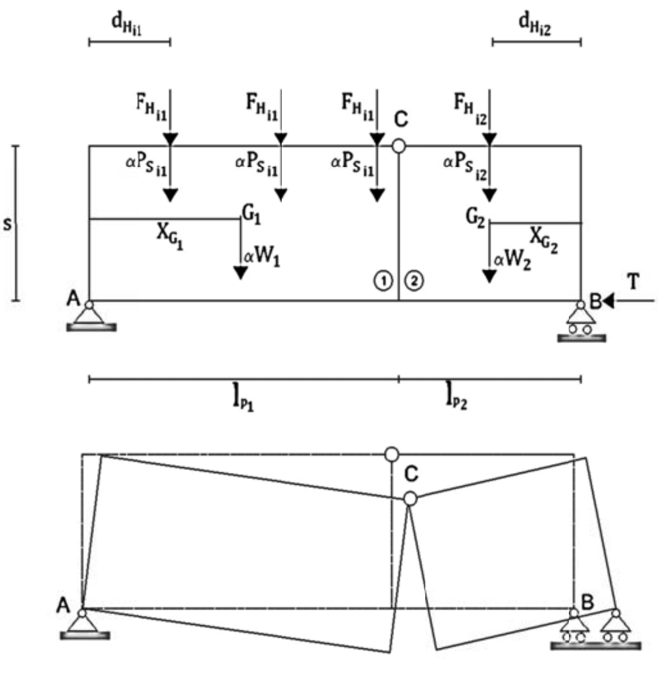

Figure 6 - *Horizontal bending, horizontal section of the wall*

4 Results of the simplified analysis

The simplified structural analysis performed by combining the limit state conditions reported in Section 3 has been performed considering 100,000 randomly generated virtual buildings which are modelled by means of macro elements. Moreover, a preliminary analysis of the typological features permits to attribute a vulnerability class to each structural model. According to the SAVE methodology, four classes A-D are defined spanning from high vulnerability (class A) to low vulnerability (class D). The analysis procedure performed for

each virtual building consists in computing the minimum value a_g of the horizontal acceleration for which any of its macro elements attains one of the limit state conditions corresponding to the kinematic schemes reported in Section 3.
To fix ideas, Table 2 summarizes some occurrences of the virtual building dataset analysed in this research. In particular, the first two columns represent, respectively, a progressive building identification number and the vulnerability class assigned to the model. Moreover, columns 3-8 report the values of the trigger acceleration for which the weakest panel of the model attains at the corresponding ultimate limit state. Finally, assuming a series system behaviour, column 9 reports the minimum value of the trigger acceleration for which the global collapse of the model is assumed. Results of the simplified structural analyses are analysed from a statistical point of view. Grouping the buildings by vulnerability class, the arithmetic mean value and the standard deviation of the $a_g$values are calculated for each class and for all the mechanisms. These values consist in the first two moments of the Damage State Probability distributions characterizing the attainment of each limit state.
In general, the values of the first two moments are not exhaustive of the limit state statistics. Nevertheless, a widely accepted assumption consists in modelling damage state probability curves by means of lognormal distributions which can be exhaustively characterized by the computed mean and variance. Probability curves related to each collapse mechanism and vulnerability class are reported in Figures 7-10 where the collapse probability is plotted as function of the ground acceleration a_g. Building typology determining the vulnerability class turns out to have a great influence on the probability distribution trend. In particular, class A buildings, whose curves are reported in Figure 7, present high vulnerability with respect to the shear crack and overturning collapse since these curves are shifted to the left of the graph with respect to the remaining distributions. This result is consistent with expectations since shear vulnerability is related to the poor mechanical strength of the masonry belonging to class A. Moreover, such a building typology presents masonry walls without strong mutual connection; for this reason, overturning mechanisms are likely to be activated even for low values of the base acceleration.

Table 2 - *Value of the trigger acceleration (a_g) corresponding to the considered collapse mechanisms (sample of the first 8 occurrences of the computed dataset)*

ID	Buildings Classes	In-plane mechanisms			Out of plane mechanisms			MIN
		Shear Crack	Buckling failure	Failure by slip	Vertical Bending	Horizontal Bending	Simple Overturning	
1	A	0,17	0,50	0,42	0,44	1,27	0,37	0,17
2	A	0,32	0,24	0,34	0,38	0,16	0,30	0,16
3	C	0,10	0,28	0,25	0,46	0,70	0,37	0,10
4	B	0,83	0,55	0,60	0,59	0,79	0,66	0,55
5	D	0,51	0,43	0,35	0,22	0,69	0,68	0,22
6	A	1,50	0,56	0,38	0,40	0,86	0,72	0,38
7	D	0,12	0,25	0,40	0,65	0,42	0,54	0,12
8	C	0,29	0,37	0,34	0,25	0,26	0,21	0,21
...	...	...	...	....	...	..	...	...
100,000	A	0,36	0,43	0,52	1,03	1,13	1,18	0,36

Structures belonging to vulnerability class B, shown in Figure 8, present wall connections and a higher mechanical strength in fact probability distributions turn out to be shifted to the right of the graph with respect to the class A curves. Nevertheless, shear and overturning collapse mechanisms remain the governing limit states of such a structural typology.

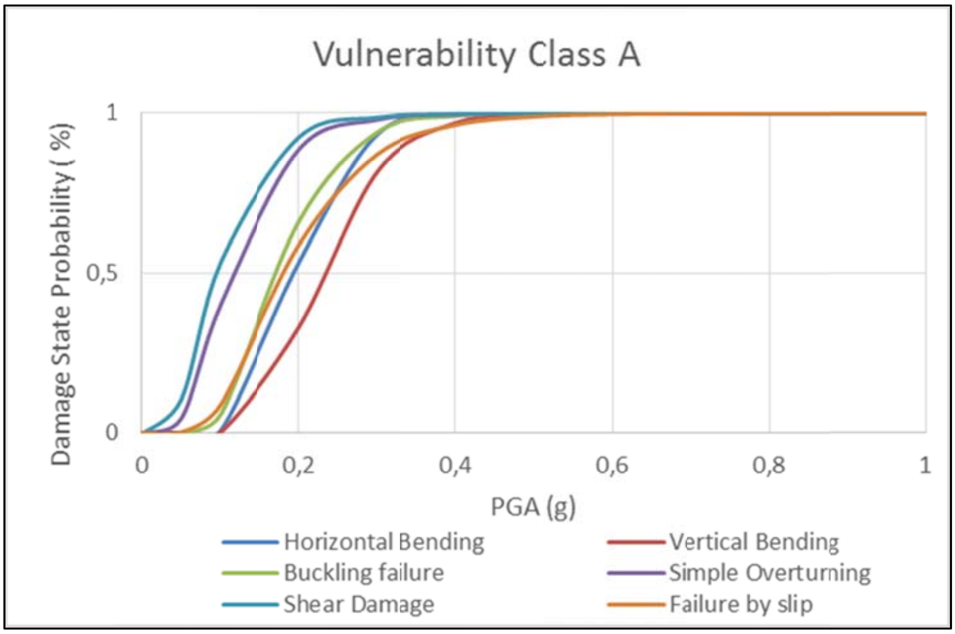

Figure 7 - *Lognormal distribution curves for buildings vulnerability class A as functions of trigger acceleration of each Collapse Mechanism (PGA) vs the Damage State probability*

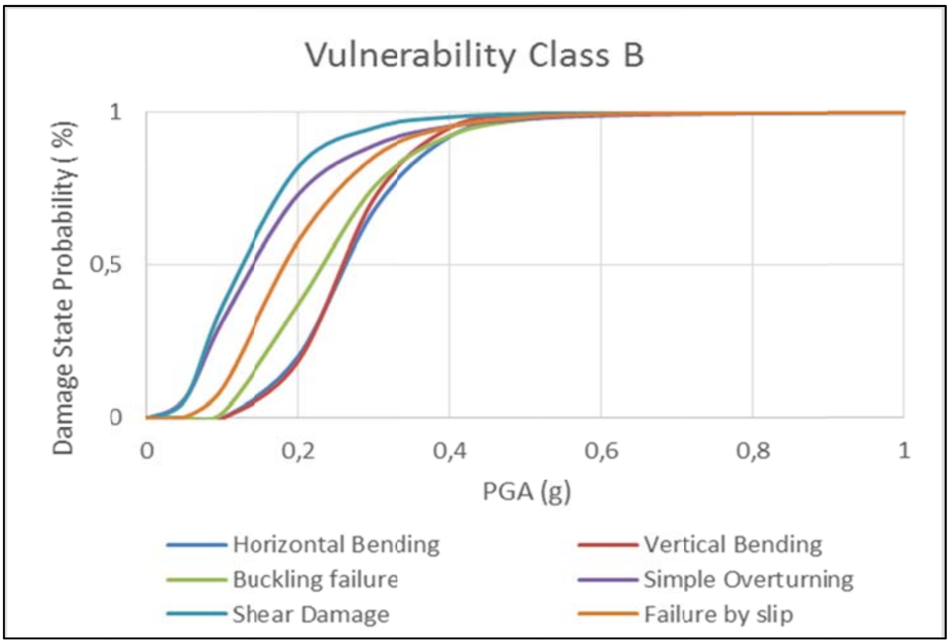

Figure 8 - *Lognormal distribution curves for buildings vulnerability class B as functions of trigger acceleration of each Collapse Mechanism (PGA) vs the Damage State probability*

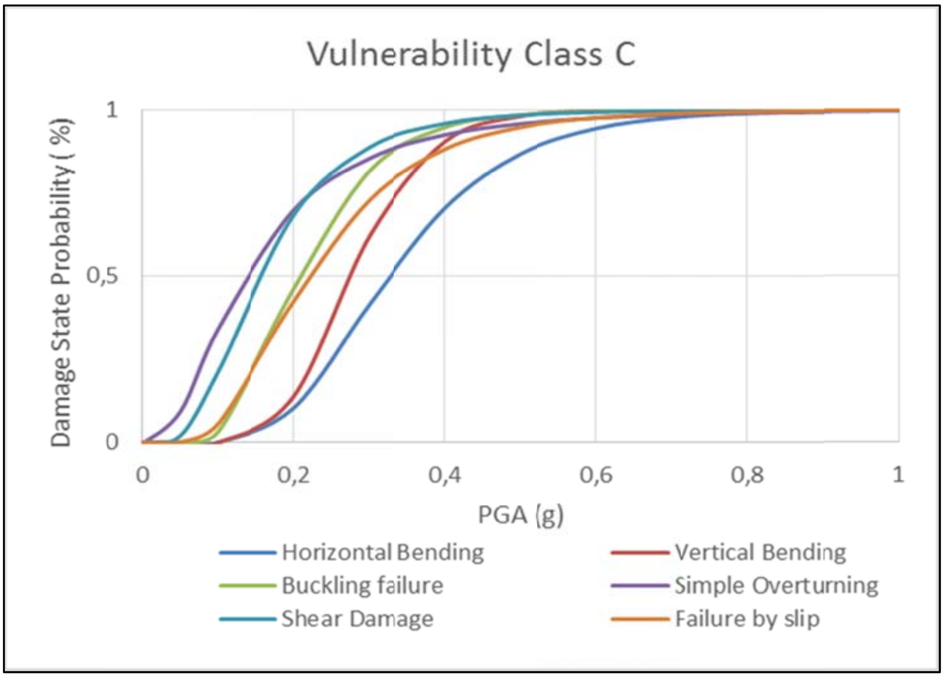

Figure 9 - *Lognormal distribution curves for buildings vulnerability class C as functions of trigger acceleration of each Collapse Mechanism (PGA) vs the Damage State probability*

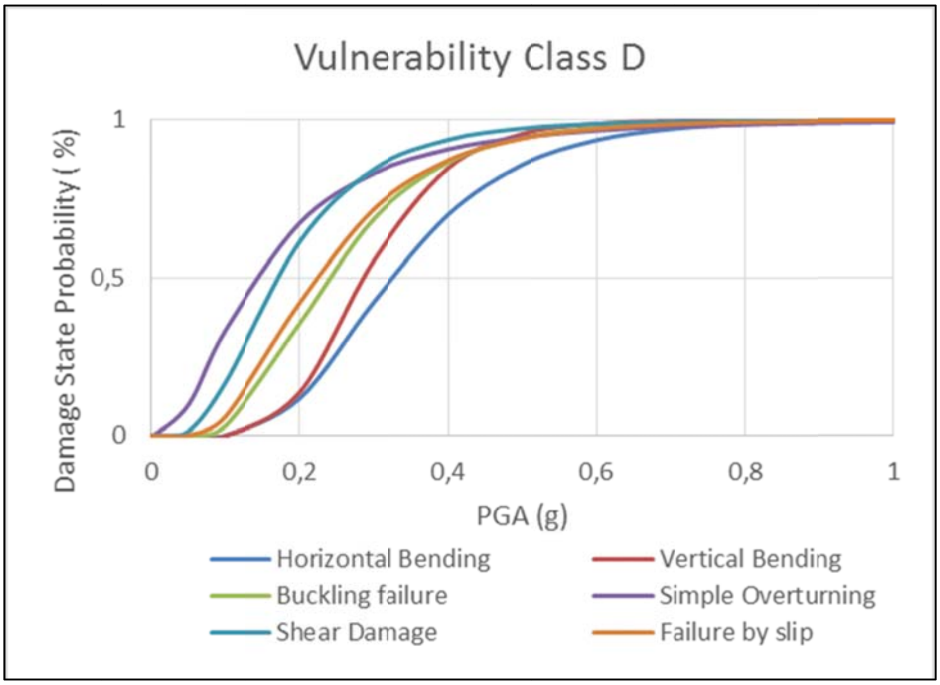

Figure 10 - *Lognormal distribution curves for buildings vulnerability class D as functions of trigger acceleration of each Collapse Mechanism (PGA) vs the Damage State probability*

Probability distributions corresponding to classes C and D, shown in Figures 9 and 10, respectively, present a further shifting to the high accelerations meaning that the collapse probability is decreasing as long as the structural strength is improved. In particular, while at low values of the trigger acceleration influence of shear and overturning on structural

collapse is still significant, at high acceleration values, corresponding to the tails of the probability distributions, the remaining limit states present higher collapse probability so that the global failure is ruled by a combination of all the considered mechanisms.
Further considerations can be made by analysing the predominant collapse typology. Figure 11 presents, for each vulnerability class, the percentage of the structural models belonging to the virtual dataset collapsed either for in-plane or out-of-plane mechanisms. From a frequentistic point of view, and assuming that the dataset is significant of the Italian building asset, the graph reports the probability that the failure of a structure belonging to a specific class, if occurs, is expected to be determined by either in-plane or out-of-plane collapses.
In general, the probability of out-of-plane phenomena decreases proportionally to the vulnerability and, in the case of classes B, C and D does not present significant differences since the probability of out-of-plane phenomena is about 40%. On the contrary, collapse of class A buildings has the 60% probability to be induced by out-of-plane collapses.
This is an important aspect from a resilience point of view; in fact, out-of-plane failures turn out to be more disastrous than in-plane ones. In this sense, retrofit of out-of-plane collapses are far more difficult than the repair of in-plane damages and are expected to arouse significant casualties because of their rapid progression.

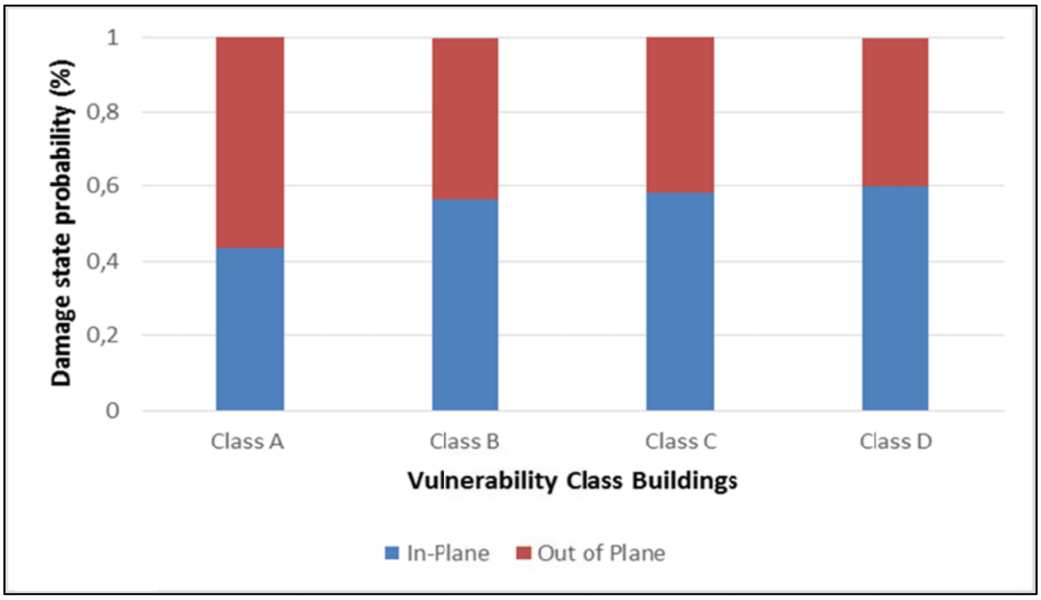

Figure 11 - *Mechanism type activating probability for each vulnerability class (A, B, C, D)*

5 Damage vulnerability curves

Global vulnerability curves, reported in Figure 12, are obtained by assuming the global failure of each single structural model at the attainment of the first limit state condition. Thus, global distributions are computed by assuming lognormal trend and computing, for each vulnerability class, mean and standard deviation of the ground acceleration minimum values reported in the last column of Table 2.
It is worth to emphasize that global curves are not merely an envelope of the single-mechanism distributions reported in Figures 7-10. On the contrary, the computation of the first two probabilistic moments of a_g is capable of accounting for the mechanisms' correlation.

In this sense, global failure curves represent the joint probability that any of the collapse mechanisms is triggered by the ground acceleration. For convenience, a further representation of structural vulnerability is shown in Figure 13 where global failure probability is represented as function of a macro-seismic intensity index computed by the equation proposed by Faccioli and Cauzzi [2006]. Although conceptually probabilities reported in Figures 12 and 13 have the same physical sense, the plotted curves apparently have different trends. This issue is due to the fact that relationship between collapse ground acceleration and seismic intensity is, in general, non-linear. Nevertheless, both these representations are useful in order to compare the results of the proposed strategy with vulnerabilities computed by alternative procedures.

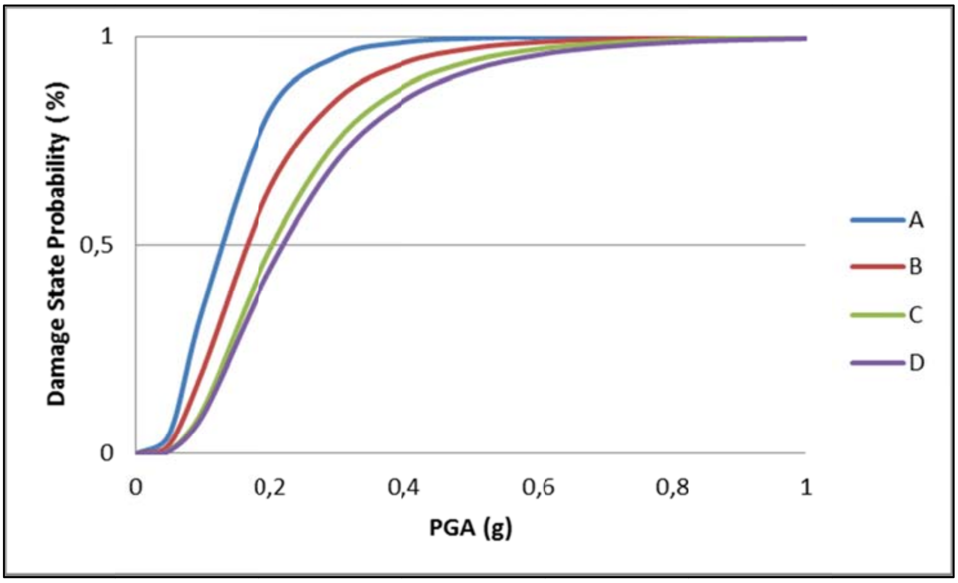

Figure 12 - *Damage Vulnerability Curves for each vulnerability Class (A, B, C, D)*

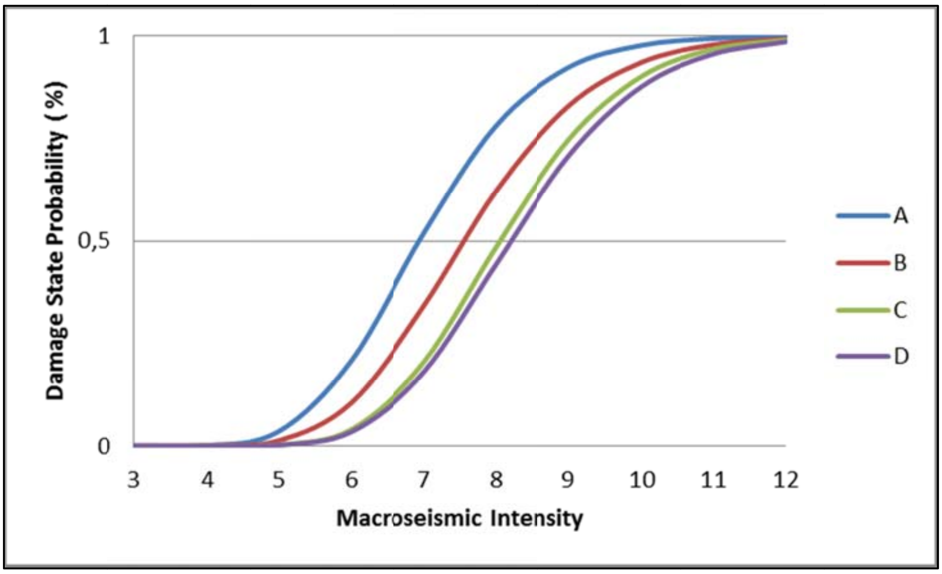

Figure 13 - *Damage Vulnerability Curves for each vulnerability Class (A, B, C, D)*

6 Comparison with alternative strategies

In order to investigate the accuracy of the proposed methodology, the probability curves reported in the previous section are compared with the distributions obtained by the application of alternative strategies available in the literature and, in particular, applying both a mechanical and a statistical approach.

The mechanical approach has been used by Cattari et al. (2014) which developed fragility curves for vulnerability class B then extrapolated to the other classes. Comparison with the proposed strategy are reported in Figures 14 and 15 where probability curves have been plotted, respectively, with respect to the ground acceleration and the seismic intensity.

In both the cases, the proposed strategy presents a good matching with the curves esteemed for a damage level defined as DS3 which denotes the first significant structural damage occurred in the structural elements which does not necessarily implies the contemporary collapse of several structural elements. This is consistent with the assumptions of the proposed strategy where vulnerability is function of the first limit state attainment. In fact, partial and total collapse of the structure, corresponding to damage levels D4 and D5, usually occur when more than a single structural element is collapsed, especially in presence of in-plane mechanisms. In this sense, the proposed strategy proved to be encouragingly consistent with the benchmark data.

The partial or total collapse of the buildings (D4-D5), that generally requires not only the trigger of the first mechanism but the occurrence of more mechanisms and greater displacements, will be taken into account in the following development of the research.

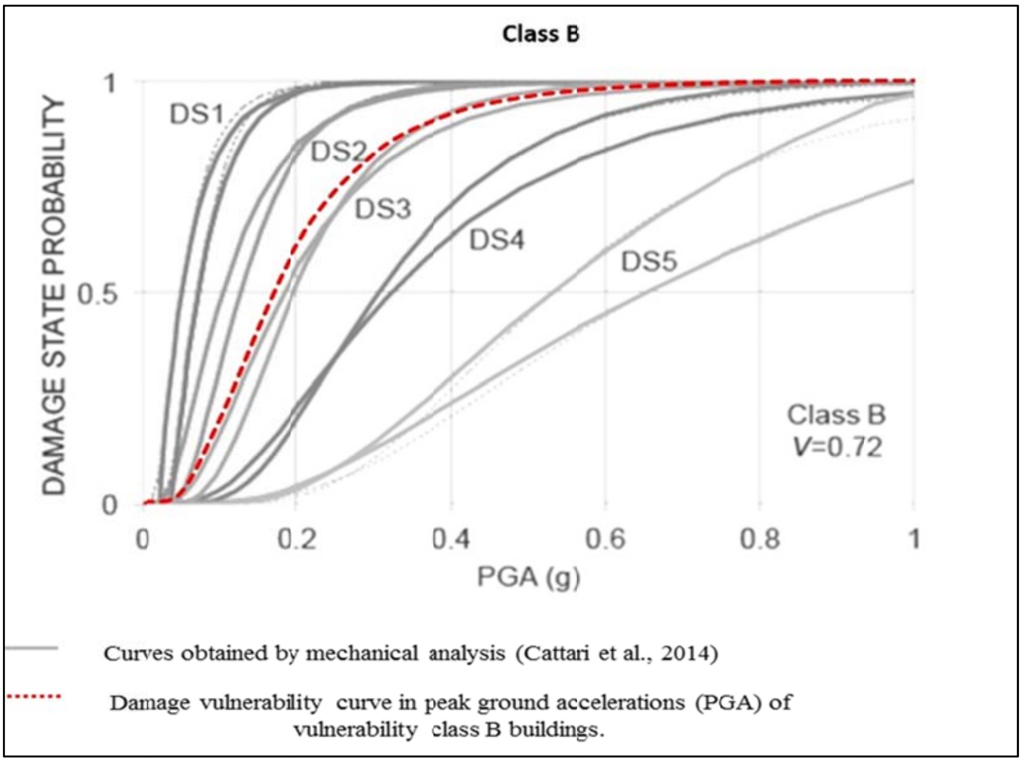

Figure 14 - *Comparison between damage Vulnerability Curves of vulnerability Class B and the one obtained by mechanical analysis [Cattari et al, 2014]*

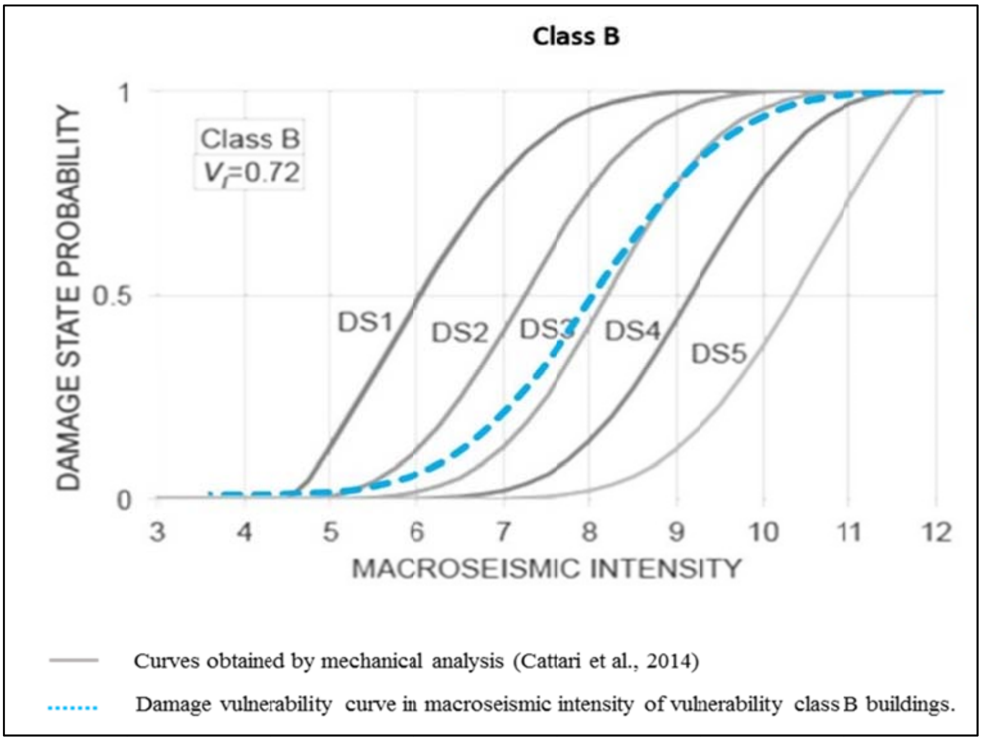

Figure 15 - *Comparison between damage Vulnerability Curves of vulnerability Class B and the one obtained by mechanical analysis [Cattari et al, 2014]*

The vulnerability curves are compared also with the empirical curves derived from the Damage Probability Matrices (DPM) (Table 3) [Zuccaro et al., 2015] obtained by statistical fitting of observed damages recorded of all the past seismic events in Italy from Irpinia 1980 earthquake up to L'Aquila event in 2009. The set of statistical curves relevant to class A models, shown in Figures 16 and 17, encompasses the distributions esteemed by the proposed procedure. In particular, those latter span in between damage states D1 and D4. This is significant of the fact that, for the case of poor structural quality, even small damage mechanisms, incapable of producing significant damage for different structural typologies, could generate serious failures to vast parts of the structure. Moreover, it is worth to emphasize that, while in-plane mechanisms are limited to D1-D3 damage levels, out-of-plane curves reach the D4 level. This confirm the physical interpretation for which out-of-plane mechanisms are premonitory of serious and extended structural failures. In vulnerability class C and D (Figures 20-21, Figures 22-23) the in-plane mechanism curves are representative of a level of damage D1-D2 and out of plane curves are representative of a level of damage D1-D3. As long as the structural robustness is increased, as in the case of class B reported in Figures 18 and 19, the proposed probability distributions shift to the left side of the graphs and are encompassed by statistical curves relevant to damage states D1 and D3. Results of vulnerability classes C and D provide similar probability distributions. For both the cases, probability relevant to in-plane mechanisms are encompassed by statistical curves of damage levels D1-D2 while the out-of-plane distributions attain at the D3 level. This is due to the fact that, in presence of connecting devices and sufficient ductility, damages induced by in-plane mechanisms are usually limited to a few structural elements and do not necessarily trigger global failure.

In conclusion, comparison with both statistically and mechanically derived vulnerability curves shows a good matching of the probability distributions computed by the proposed strategy, although further investigations are required. Nevertheless, the reported comparisons have shown a sufficient robustness of the proposed procedure and confirm the physical assumptions introduced in the computational framework.

Table 3 - *DPM obtained through a statistical analysis of the data collected about the observed damages due to earthquakes occurred in Italy since 1980 [Zuccaro and De Gregorio, 2015].*

Building Class	INTENSITY	D0	D1	D2	D3	D4	D5
A	V	0,3487	0,4089	0,1919	0,0450	0,0053	0,0002
B		0,5277	0,3598	0,0981	0,0134	0,0009	0,0000
C		0,6591	0,2866	0,0498	0,0043	0,0002	0,0000
D		0,8587	0,1328	0,0082	0,0003	0,0000	0,0000
A	VI	0,2887	0,4072	0,2297	0,0648	0,0091	0,0005
B		0,4437	0,3915	0,1382	0,0244	0,0022	0,0001
C		0,5905	0,3281	0,0729	0,0081	0,0005	0,0000
D		0,7738	0,2036	0,0214	0,0011	0,0000	0,0000
A	VII	0,1935	0,3762	0,2926	0,1138	0,0221	0,0017
B		0,3487	0,4089	0,1919	0,0450	0,0053	0,0002
C		0,5277	0,3598	0,0981	0,0134	0,0009	0,0000
D		0,6591	0,2866	0,0498	0,0043	0,0002	0,0000
A	VIII	0,0656	0,2376	0,3442	0,2492	0,0902	0,0131
B		0,2219	0,3898	0,2739	0,0962	0,0169	0,0012
C		0,4182	0,3983	0,1517	0,0289	0,0028	0,0001
D		0,5584	0,3451	0,0853	0,0105	0,0007	0,0000
A	IX	0,0102	0,0768	0,2304	0,3456	0,2592	0,0778
B		0,1074	0,3020	0,3397	0,1911	0,0537	0,0060
C		0,3077	0,4090	0,2174	0,0578	0,0077	0,0004
D		0,4437	0,3915	0,1382	0,0244	0,0022	0,0001
A	X	0,0017	0,0221	0,1138	0,2926	0,3762	0,1935
B		0,0313	0,1563	0,3125	0,3125	0,1563	0,0313
C		0,2219	0,3898	0,2739	0,0962	0,0169	0,0012
D		0,2887	0,4072	0,2297	0,0648	0,0091	0,0005
A	XI	0,0002	0,0043	0,0392	0,1786	0,4069	0,3707
B		0,0024	0,0284	0,1323	0,3087	0,3602	0,1681
C		0,0380	0,1755	0,3240	0,2990	0,1380	0,0255
D		0,0459	0,1956	0,3332	0,2838	0,1209	0,0206
A	XII	0,0000	0,0000	0,0000	0,0010	0,0480	0,9510
B		0,0000	0,0000	0,0006	0,0142	0,1699	0,8154
C		0,0000	0,0001	0,0019	0,0299	0,2342	0,7339
D		0,0000	0,0002	0,0043	0,0498	0,2866	0,6591

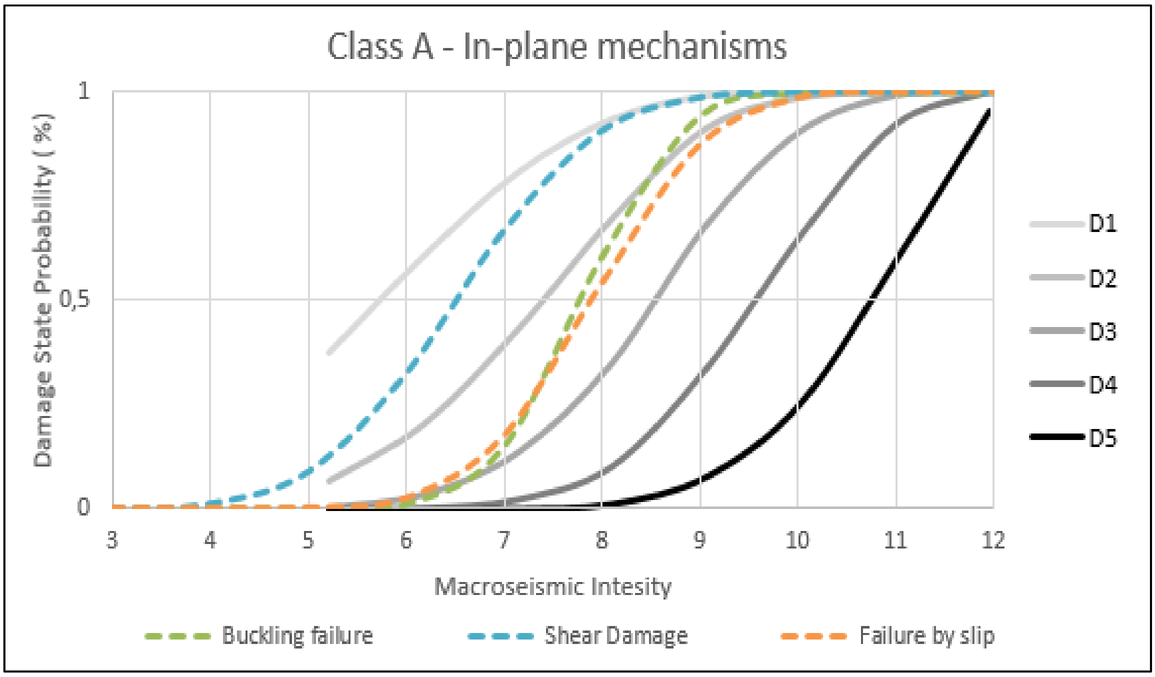

Figure 16 - *Comparison between the In-plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class A*

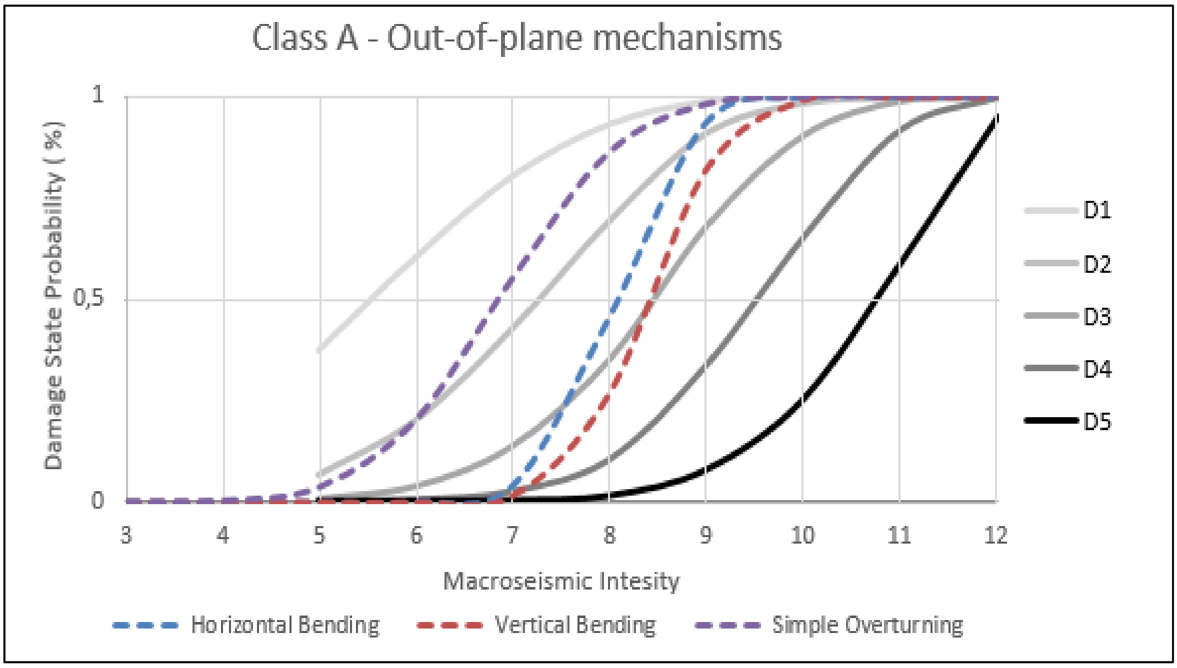

Figure 17 - *Comparison between the out of plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class A*

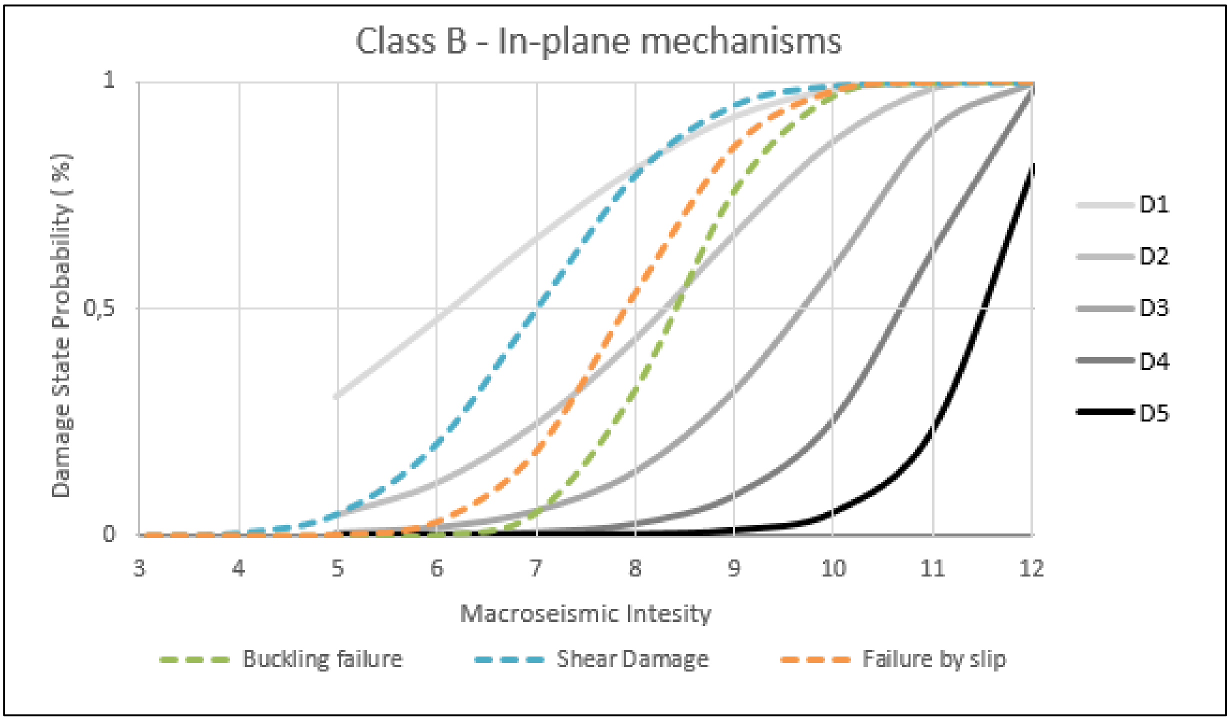

Figure 18 - *Comparison between the In-plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class B.*

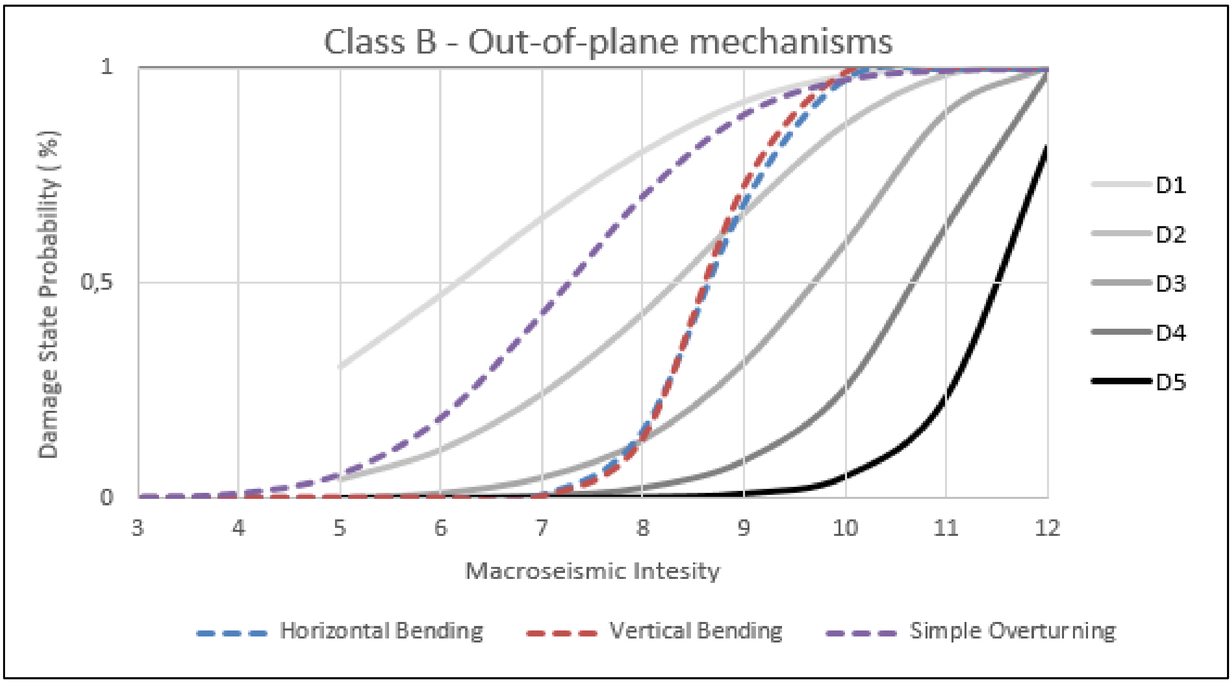

Figure 19 - *Comparison between the out of plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class B*

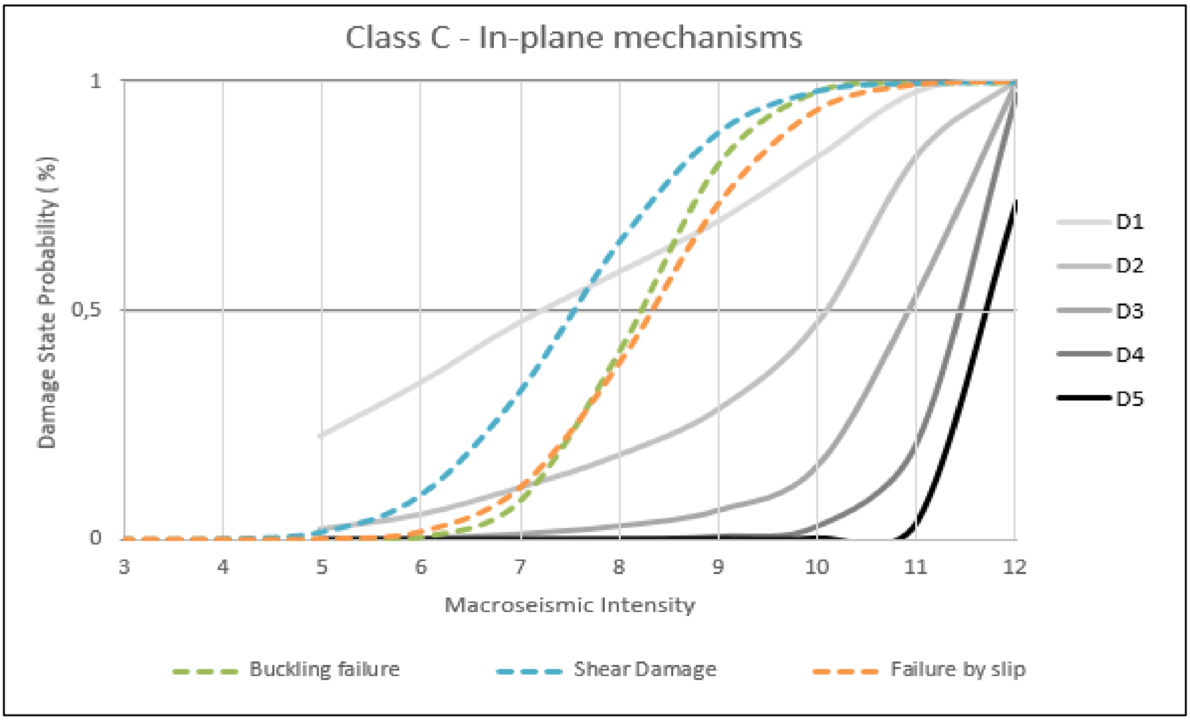

Figure 20 - *Comparison between the In-plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class C*

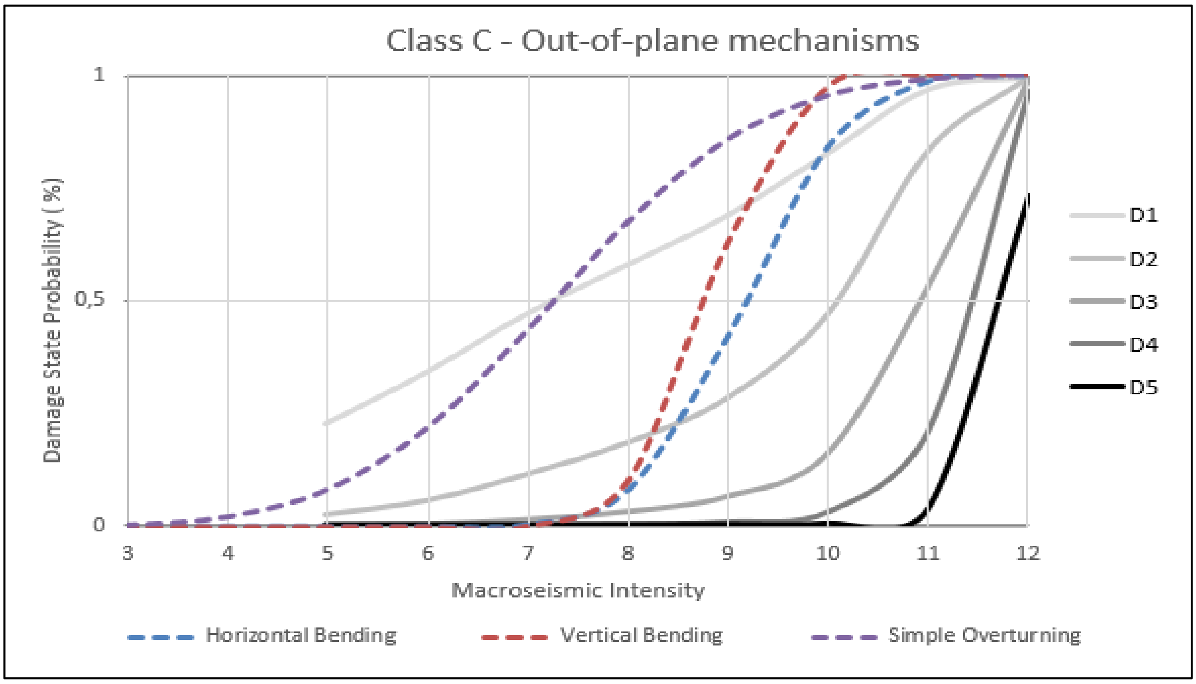

Figure 21 - *Comparison between the out of plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class C*

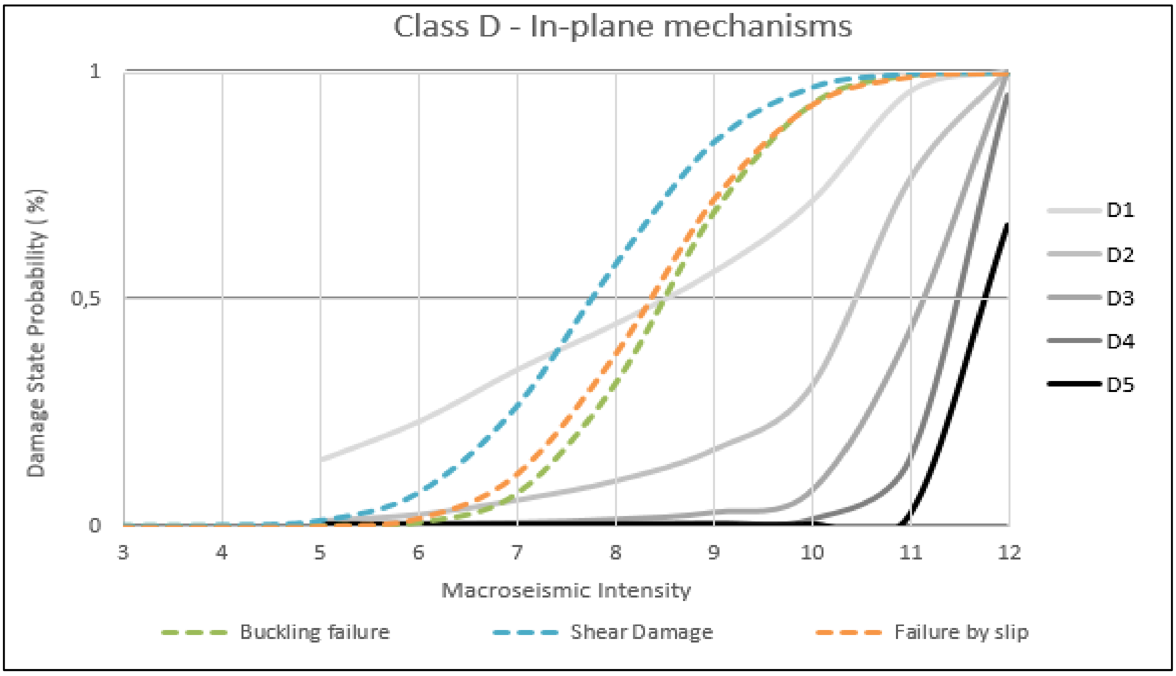

Figure 22 - *Comparison between the In-plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class D*

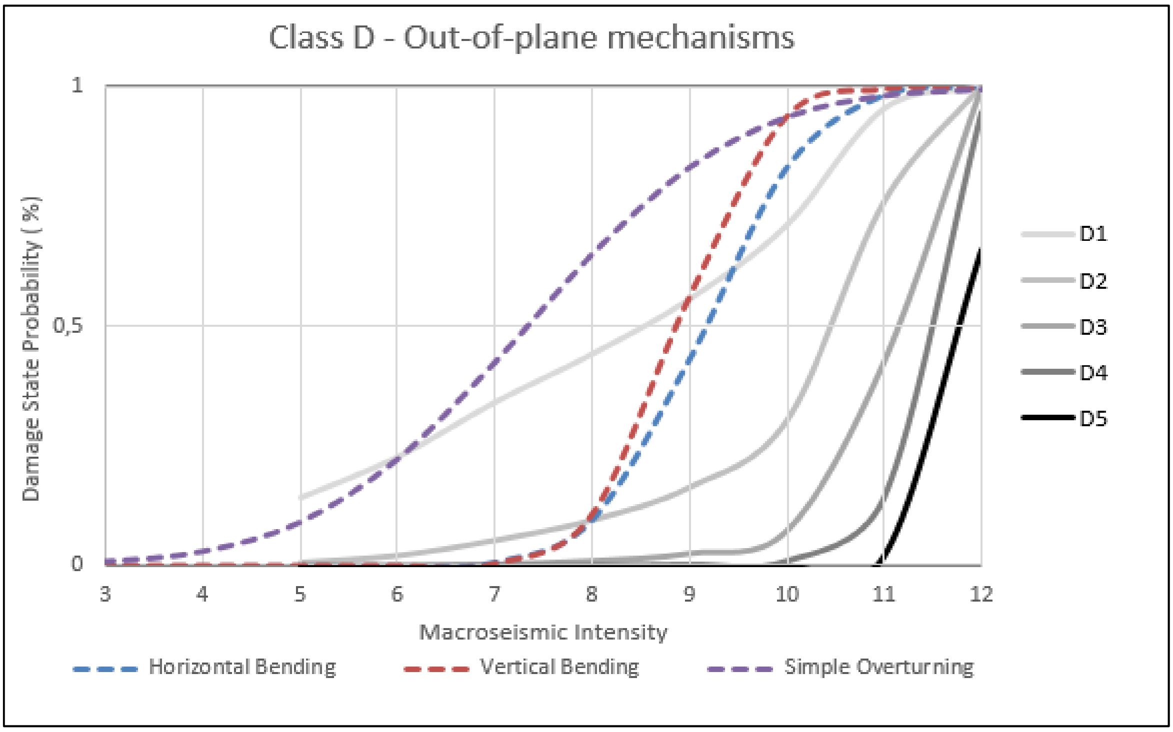

Figure 23 - *Comparison between the out of plane mechanisms vulnerability curves and vulnerability curves derived from DPM [Zuccaro and De Gregorio, 2015] for vulnerability class D*

7 Conclusions

An hybrid methodology for determining the vulnerability curves relevant to building typological categories is presented. It consists in determining a virtual set of structural models generated by random assignment of geometrical and mechanical properties extract from ranges of values parameters statistically characterized from an observed set of existing masonry buildings data base representing the Italian heritage to analyse by limit state analysis in order to get trigger acceleration values for the most common collapse mechanisms.
Each virtual structure is therefore analysed by a simplified procedure in which the structural performance is computed with respect of a limited set of in-plane and out-of-plane collapse mechanisms commonly observed in the experimental dataset.
Assuming a series system behaviour of each structural model, and in particular, setting as global failure condition the attainment of the first limit state, the probability distributions of global collapse with respect to the base acceleration have been derived.
Such distributions have been compared with benchmark vulnerability curves available in the literature and derived by both a statistical and mechanical approach.
The proposed methodology proved to be consistent with the benchmark results confirming physical interpretations commonly accepted in common practice, such as the high fragility induced by out-of-plane mechanisms. Nevertheless, some of the computed vulnerability curves turn out to be over-conservative with respect to the considered benchmarks.
Such an issue is due to the series system hypothesis assumed for the simplified structural analysis procedure. In fact, most of the considered collapse mechanisms, although compromising the structural capacity of one or more structural elements, do not necessarily induce global failure phenomena.
For this reason, further investigations are required in order to account for the correlation of limit state conditions in different structural elements, ductility and more complex global kinematics. Such further information is oriented to determine failure probability distributions not only related to structural typologies, but also significant of different damage states.
Moreover, recent earthquake events occurred in Central Italy, such as the Amatrice Earthquake in 2016, involved several masonry structures and important survey activities are undergoing. The forthcoming diffusion of more significant observed data will permit a further assessment of the proposed strategy both in defining the initial experimental dataset and in providing benchmark to be employed in the results comparison.

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ANALISI DEI MECCANISMI DI COLLASSO E TIPOLOGIE DI STRUTTURE IN MURATURA: UNA POSSIBILE CORRELAZIONE

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SOMMARIO: *Nell'ambito della valutazione del rischio a scala nazionale e regionale, l'analisi dei meccanismi di collasso indotti dalle accelerazioni sismiche costituisce un utile strumento per comprendere il comportamento delle strutture in muratura e per pianificare strategie di mitigazione ed interventi di consolidamento. In questa ottica, obiettivo del presente studio è la identificazione delle correlazioni tra tre fattori: 1) le tipologie strutturali che caratterizzano gli edifici italiani in muratura, 2) i possibili meccanismi di collasso (in piano e fuori dal piano), 3) l'accelerazione di picco al suolo. Le analisi sviluppate riguardano un campione di 100.000 modelli, che rappresentano le tipologie strutturali degli edifici ordinari in muratura distribuiti sul territorio italiano. Essi sono ottenuti dall'esame delle caratteristiche strutturali (dimensioni degli elementi strutturali, caratteristiche meccaniche dei materiali, tipologia degli orizzontamenti e delle coperture, presenza di volte e/o catene o cordoli, numero di piani, ecc.) raccolte attraverso rilievo in sito di circa 250.000 edifici distribuiti lungo il territorio italiano. Per ciascun modello, attraverso analisi semplificate agli stati limite, è calcolata l'accelerazione in grado di indurre il primo meccanismo di collasso. I risultati sono elaborati allo scopo di calcolare, per ciascuna classe di vulnerabilità (valutata attraverso la combinazione delle caratteristiche tipologico-strutturali), le probabilità di occorrenza di ciascun meccanismo di collasso al variare dell'accelerazione di picco al suolo (PGA).*

KEYWORDS: *strutture in muratura, vulnerabilità sismica, meccanismi di collasso.*

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