ORIGINAL ARTICLE

A mechanical-based seismic vulnerability assessment method with an application to masonry structures in Cosenza (Italy)

Francesco S. Liguori[1] · Stefania Fiore[1] · Francesca L. Perelli[2] ·
Daniela De Gregorio[2,3] · Giulio Zuccaro[2,3] · Antonio Madeo[1]

Abstract

The paper presents a mechanical-based framework for the evaluation of local-scale seismic fragility curves. The approach is oriented to a seismic vulnerability assessment of unreinforced masonry buildings and makes use of basic exposure data easily obtained from survey or available in existing database. An efficient finite element model and static nonlinear analyses are employed to assess the structural behaviour. The mechanical-based fragility curves are evaluated using Monte Carlo simulations that allow to account for the uncertainties propagation. The proposed approach is tested on a case-study regarding the city centre of Cosenza, in southern Italy, using exposure information available from CARTIS database.

1 Introduction

The severe damages produced by recent earthquakes have demonstrated the high seismic vulnerability of the Italian territory, with particular concern on masonry structures. Therefore, growing attention has been drawn to assessing the seismic vulnerability of the building heritage, in order to address private and public sources toward mitigation strategies of the effects of earthquakes through proper seismic retrofitting strategies (Zuccaro and De Gregorio 2019; United Nations 2015; Poljansek et al. 2019; Dolce et al. 2021).

✉ Francesco S. Liguori
  francesco.liguori@unical.it

[1] Department of Computer Science, Modeling, Electronics and Systems Engineering (DIMES), University of Calabria, Ponte P. Bucci Cubo 42B, 87036 Rende, Cosenza, Italy

[2] Plinivs Studies Center-Lupt, University of Naples "Federico II", Via Toledo, 402, 80134 Naples, Italy

[3] Department of Structures for Engineering and Architecture (DiSt), University of Naples Federico II, Naples, Italy

Empirical methods have been the first vulnerability assessment methods to be studied and implemented (Whitman et al. 1973; Braga et al. 1986; Benedetti et al. 1988). They define statistical correlations among typological building characteristics, damage level and hazard values, based on information collected from past seismic events (Colombi et al. 2008; Bernardini et al. 2007; Buratti et al. 2017; Perelli et al. 2019).

Despite their widespread use, empirical methods keep some disadvantages. In particular, they suffer from localisation of data, which are available only in some locations, for some seismic intensities and for some structural typologies (Polese et al. 2008). Therefore, empirical methods are, by their nature, only useful to obtain information for large-scale planning. As a consequence, the development of new methodologies for the seismic vulnerability analysis at local level (municipal or sub-municipal), becomes the current challenge (Lamego et al. 2017; Sandoli et al. 2022; Tocchi et al. 2022). In this context, mechanical-based methods represent an interesting alternative (D'Ayala and Speranza 2003; Valluzzi et al. 2004; Borzi et al. 2008; Rota et al. 2010; Lagomarsino and Giovinazzi 2006; Abo-El-Ezz et al. 2013; Frankie et al. 2013; Asteris et al. 2014; Simões et al. 2015; Furtado et al. 2016). These methods, in fact, are based on the evaluation of the seismic vulnerability by employing mechanical models of the structures and perform numerical analyses to obtain the expected structural response under the seismic actions (Rossetto and Elnashai 2005).

Uncertainties, that are intrinsically present in empirical methods, need to be explicitly considered and propagated in mechanical models. In fact, random and epistemic uncertainties affect the geometry, the mechanical properties, the seismic action, the damage grades identification and many other aspects involved in the construction of mechanical-based fragility functions (Pinto 2005). The propagation of the uncertainties can be obtained using a full-blown Monte Carlo approach (Cosenza et al. 2005; Polese et al. 2012). Consequently, the structural analyses need to be repeated multiple times by randomly varying the parameters affected by uncertainties. Since this approach can become computationally expensive, different solutions have been proposed in which Monte Carlo simulations are combined with other methods (Lee and Mosalam 2005; Baker and Cornell 2008), or by making use of Markovian chains of conditional probability (Yang et al. 2009). However, the computational effort can still be prohibitive. Therefore, in vulnerability analyses focus should be given at reducing the computational cost of a single analysis. This can be achieved by employing structural models and analyses that balance the accuracy and the efficiency. Masonry structures, for instance, can be modelled by using simplified analytical solutions (Zuccaro et al. 2017) or more advanced models that make use of the Finite Element Method (FEM). In this last case, great accuracy is achieved by micro-models which describe the single component of the walls (brick, mortar and their interface) and take into account the distribution of the mortar joints (Lourenco and Rots 1997; Formica et al. 2002). On the other hand, macro-models are also adopted to reduce the computational effort (Sangiorgio et al. 2020). Such models consider the masonry as an homogenised material and can be applied to discretise the structure into three-dimensional, bi-dimensional or one-dimensional Finite Elements (FE). In this last group one finds equivalent frame approach that was formulated for the global response of masonry building in order to allow three-dimensional modelling by using beam elements (Lagomarsino et al. 2013). Simpler models are usually adopted and appreciate in vulnerability assessment because can take into account the main typical features of the structural response but keep simple formulation and guarantee speed execution of the analysis.

Once the structure is discretised into FEs, different methods are available to obtain the seismic intensity that produces each damage level. To this end, one can adopt Incremental Dynamic Analyses (IDA) (Vamvatsikos and Cornell 2002). It is composed by a sequence

of dynamic-nonlinear analyses in which the seismic action is described as ground motion time-history excitations of growing intensity. Despite its high computational cost, IDA has been adopted for vulnerability analysis, among the others, of regular and irregular concrete and steel buildings (Fadzli Mohamed Nazri and Saruddin 2017). Recently, nonlinear dynamic analyses have been employed in the seismic fragility assessment towards local mechanisms (Nale et al. 2020) Even if IDA represents a complete and accurate approach in seismic analysis, its use is too computationally expensive when many buildings need to be analysed. This happens, for instance, when fragility curves need to be obtained for buildings on a specific area.

The static nonlinear (pushover) analysis gives a valid and efficient alternative. It allows the post-elastic regime to be investigated by keeping the advantages of static analyses. The seismic action on the building is applied by means of distributions of static horizontal forces that increase their amplitude up to structural collapse. The result is a capacity curve that represents the structural response path of equivalent single-degree-of-freedom oscillator. Thanks to their efficiency, static nolinear analyses are widely used in vulnerability assessment methods (Seyedi et al. 2010; Polese et al. 2012; Nettis et al. 2021; Rossetto and Elnashai 2005).

Different choices of modelling and analysis techniques have led to the formulation of many seismic vulnerability methods for masonry structures. Lagomarsino and Cattari (2014) proposed a mechanical model for the derivation of fragility curves based on a simplified model of the in-plane behaviour calibrated on an equivalent frame model. Uncertainties of different nature are considered as independent and propagated using the response surface method. A mechanical vulnerability assessment strategy using Monte Carlo simulations is proposed by Rota et al. (2010). A combination of pushover and nonlinear dynamic analyses is employed to define the fragility functions. Kappos et al. (2006) proposed a hybrid method in which empirical data are used in conjunction with pushover analyses on equivalent frame buildings to construct fragility functions. Recently, an approach based on the vulnerability index and static nonlinear analyses and an equivalent frame of the masonry structure has been proposed (Nikolic et al. 2021). Fragility functions which consider the out-ofplane behaviour of masonry structures using a failure mechanism approach are presented by D'Ayala (2005); Zuccaro et al. (2017). The automated mechanical vulnerability analysis of masonry aggregates has been recently proposed by Leggieri et al. (2021).

In this work, a mechanical-based vulnerability assessment method, named HAREAS (Hybrid Analysis for the REgional-scale Assessment of Seismic vulnerability) is proposed. The main purpose is to provide a framework for developing seismic fragility curves at sub-municipal scale and valid for specific masonry structures starting from a rapid and automatic exposure analysis. The mechanical model adopted is implemented in the commercial software POR-2000 (Newsoft 2021) and is chosen to provide a compromise between accuracy and efficiency. It is based on a bidimensional discretisation using a hybrid-stress FE that captures the key features of the global behaviour of masonry structures at a reduced computational cost. The global structural behaviour is obtained by performing static nonlinear analyses for varying directions and shape of the seismic action. The use of many seismic directions allows to increase the range of validity of pushover analyses to irregular buildings. The damage produced by local collapse mechanisms is also considered using simplified kinematic analyses.

The procedure starts from the knowledge of the building heritage. In particular, information about the structural features are extracted from CARTIS database which collects specific data for many Italian municipalities (Zuccaro et al. 2015). CARTIS database

information allows to develop a more reliable exposure assessment than the more consolidated approaches for the Italian seismic vulnerability assessment (Zuccaro et al. 2015) that adopted the information from the Census data (vertical structure, age and number of floors) in aggregate form across the entire country. Thanks to its features, CARTIS form has been recently used in medium-large territorial scale vulnerability analyses (Olivito et al. 2021; Formisano et al. 2021; Sandoli et al. 2022; Leggieri et al. 2021; Chieffo and Formisano 2019a, b, 2020; Chieffo et al. 2021).

The collected data is used to generate statistical distributions of the typological features involved in the vulnerability assessment. Additionally, uncertainties on geometry and mechanical properties, on the response spectrum shape and on the identification of the damage grades are considered. Thanks to the high efficiency of the employed mechanical model, a full Monte Carlo simulation is adopted. Each generated building is subject to global and local analysis, thereby obtaining the evolution of its static and kinematic configuration with PGA. These results are heuristically correlated with the damage scale reported in the European Macroseismic Scale 1998 (EMS-98) (Grünthal 1998).

The fragility curves are constructed for the vulnerability classes obtained by applying the SAVE method (Zuccaro and Cacace 2015) based on the analysis of different typological-structural characteristics and statistical correlations derived from the observation of the damage produced by previous earthquakes. In parallel, the proposed methodology can be used to directly evaluate urban fragility curves (Sandoli et al. 2022; Tocchi et al. 2022) in order to provide a global indicator of the structural vulnerability of a sub-municipal area.

The proposed approach is applied to the vulnerability assessment of the masonry structures in the city centre of Cosenza, in southern Italy. This case-study is characterised by buildings built in the 1920s–1930s in which the predominant structural typology is the irregular masonry. An in-situ survey is conducted for 50 buildings and the simulated sample generated by the Monte Carlo simulation contains thousands of structural models. Two vulnerability classes are identified. The first one corresponds to class A of EMS-98 and is characterised by high-vulnerability buildings lacking anti-seismic features. The second one is between classes B and C of EMS-98 and shows lower damage at high intensities. The obtained results are compared with empirical fragility curves. Additionally, results highlight that both global and local responses strongly influence the fragility curves.

The paper is organised as follows. The exposure analysis and the Monte Carlo generation is shown in Sect. 2. The vulnerability classes assignment using SAVE is briefly recalled in Sect. 3. Some details on the numerical analyses of masonry structures are given in Sect. 4. The evaluation of the mechanical-based fragility curves is presented in Sect. 5. In Sect. 6 the case study of Cosenza city centre is presented. Finally, conclusions are drawn in Sect. 7.

2 Exposure model and Monte Carlo generation

This section presents the first part of the proposed HAREAS method for the vulnerability assessment of masonry structures at a sub-municipal scale. First, a survey activity is conducted to obtain information on the building under consideration. Then, an exposure model is constructed through a statistical elaboration of the data. Finally, the Monte Carlo method is used to construct a virtual population of structures which reflects the significant features of the analysed buildings.

2.1 Two-levels survey activity

The survey activity is conducted in two different levels. The first level is conducted for all the buildings under consideration and concerns easy obtainable data. Conversely, the second level is conducted for a restricted number of buildings and is aimed at obtaining more specific information.

2.1.1 First level survey

The first level survey is aimed at identifying for all the buildings plan shape and dimensions, number of storeys, presence of underground levels and irregularity in elevation.

Plan shapes and dimensions are detected automatically within a GIS environment. For doing so, the starting point is represented by a technical map of the district where the buildings are represented as polylines. The key idea is to recognise known shapes among the actual building plans in order to have simplified geometries to be used in the subsequent Monte Carlo generation algorithm. In particular, some shape types are considered, namely rectangular or irregular "T-shaped", "L-shaped", "O-shaped" and "C-shaped" and denoted with *R, T, L, O, C,* respectively, each one characterised by parametric lengths collected in the vector **L**, as shown in Fig. 1. Then, for each building, the shape that best fits the actual plan is sought by solving an optimisation problem. For convenience, the polygon which represents the actual plan is represented by its turning function, denoted as $\Theta_a[s]$, being s a curvilinear abscissa along the polygon. Analogously, the approximant, unknown polygon is described by the turning function $\Theta_b^S[s, \mathbf{L}]$, where S identifies the predefined shape. The best fitting plan is therefore obtained by solving a minimisation problem that reads as

$$\min_{L,S} \quad \begin{aligned} & d_2[\Theta_a[s], \Theta_b^S[s, \mathbf{L}]] \\ & \min[\mathbf{L}] \geq 0 \end{aligned} \tag{1}$$
$$\text{subject to} \quad \begin{aligned} & S = R, T, L, O, C \\ & \|A_b - A_a\| \leq \varepsilon \end{aligned}$$

where A_a and A_b are the area of the approximant and of the actual plan, respectively, while ε is a given tolerance. The solution of the optimisation problem is easily obtained with a genetic algorithm. The function d_2 is a L2 norm of the distance between two polygons evaluated according to the metric proposed by Arkin et al. (1991)

$$d_2[\Theta_a[s], \Theta_b[s]] = \min_{t \in [0,1]} \left(\int_0^1 \left(\Theta_a[s+t] - \Theta_b[s] \right)^2 ds - (\alpha - 2\pi t)^2 \right) \tag{2}$$

where $\alpha = \int_0^1 \left(\Theta_b[s] - \Theta_a[s] \right) ds$.

"R" plan   "C" plan   "O" plan   "L" plan

Fig. 1 Plan shapes and dimensions

The norm is invariant under translation, rotation and change of scale.

Another information that can be automatically obtained is the building height. In fact, it is evaluated as difference between the Digital Surface Model and the Digital Terrain Model. Finally, it is worth noting how additional information can be automatically obtained to enrich the knowledge of the buildings adopting machine-learning approaches as proposed by Ruggieri et al. (2021).

2.1.2 Seconds level survey

The second level survey is conducted for a portion of the studied buildings and has the purpose of obtaining more detailed features affecting the seismic vulnerability. To this end, the use of the Cartis building form is proposed (Zuccaro et al. 2015). Over the last few years, with the aim of incentive studies and research activities on risk assessment at territorial scale, the Italian Department of Civil Protection, promoted the research activities of "Inventory of existing structures and building typologies Cartis". Cartis aims to furnish information collected by expert judgment at sub-municipal scale, with the aim to collect data covering a large part of the regional territory in the perspective to develop regional characterizations of Italian ordinary buildings more reliable than simple census data. The Cartis form aims at making the typological-structural characterisation of specific urban areas, called "Sectors", resulting from a careful parting of the total municipal surface. Sectors can be defined as homogeneous areas characterised by the presence of buildings with similar typological-structure and/or age of construction. In addition, an ad hoc form to collect information building by building has been also developed, called Cartis BUILDING. The second level survey extracts the following data from Cartis BUILDING form:

1. Type of masonry;
2. Slab type;
3. Thrusting roof;
4. Presence of tie-rods;
5. Inter-storey height;
6. Wall width at ground floor;
7. Percentage area of openings;
8. Mean wall length.

It is worth noting that in many Italian municipalities a significant number of structures have already been surveyed through Cartis BUILDING form and the results are available in the Cartis database. For such areas, the second level survey in HAREAS can be easily performed by extracting the information from the database and no in-situ activities are required. Conversely, all the required data can be obtained by compiling Cartis form after a rapid survey and with the support of expert judgment.

2.2 Exposure model

The two levels survey stage provides some geometrical features for all the structures under consideration and other data only for some of them. The application of HAREAS requires that an exposure model is constructed. This is obtained through a statistical elaboration of the information provided by the second level survey in order to obtain the probability functions that best fit the frequencies of occurrence for each typological feature.

2.3 Monte Carlo simulation

Starting from the exposure model for the building under consideration, a virtual sample of structures is constructed. The measures obtained in the first level survey are taken as medium value and 1 m standard deviation and a normal distribution are considered. For given plan shape and dimensions, we evaluate a probable disposition for the internal walls. The following algorithm is adopted.

- Each irregular plan shape is divided into rectangular sub domains. For instance, an "L-shaped" plan is divided into three rectangles
- Plan dimensions are modified by summing a random number that takes into account the random uncertainty in the plan measures.
- A grid is constructed in each rectangle
- The grid is obtained randomly, but imposing some constraints: (a) the total length should be preserved; (b) the average wall distance must equal the data obtained in the second level survey; (c) wall lengths should respect minimum and maximum limits.

An example of plan disposition obtained from an "L" shaped building is shown in Fig. 2.

To each wall at ground floor a thickness is assigned by random generation. Two different thickness values are considered for the internal and for the external walls. On each upper floor, it is assumed that the wall thickness reduces of 10% with respect to the lower one. Openings are inserted in the external walls and their area is such that the frequency distribution matches those obtained in the second level survey. The dimensions are the same for all the openings that are vertically aligned among floors. Slabs are inserted among all the walls. In the case of one-way slabs, the orientations are randomly assigned but are always parallel to one wall. The live load on each slab follows a normal distribution having mean value 2 kN/m^2 and standard deviation 0.5 kN/m^2. These values have been assumed to model the uncertainty in the load distribution. However, more detailed studies can be conducted to better estimate the load applied to the structures under consideration. The floor height is uniform and is extracted randomly according to

Fig. 2 Three different wall dispositions generated from a given building plan
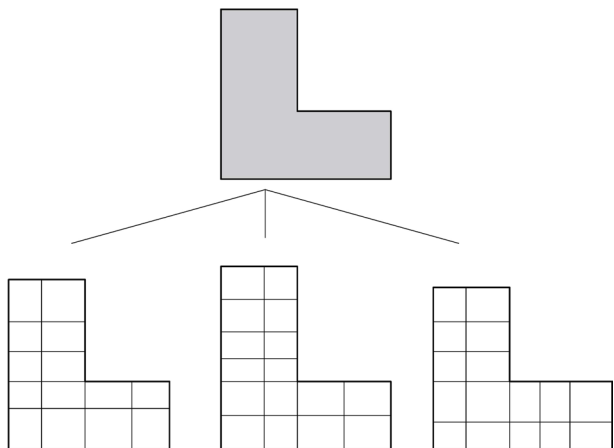
the adopted frequency distribution, except for basements that are 80% lower. In the case of irregularity in height, not all the walls are extended up to the last floor.

3 Vulnerability class assessment

The seismic vulnerability assessment using HAREAS is based on assigning a vulnerability class to each building constructed using the Monte Carlo generation. The European Macroseismic Scale 1998 (EMS-98, Grünthal 1998) is considered as a standard reference for the seismic vulnerability assessment and gives the definitions of vulnerability classes based on vertical structural typology. The EMS-98 highlights the inevitable uncertainties of class attribution, that can also significantly influence risk or impact analyses. SAVE method (Zuccaro and Cacace 2015) starts from the same concept of EMS-98 and defines the average behaviour of a building considering its vertical structure. In a second step SAVE reduces the uncertainty in the assessment of the vulnerability class through the systematic observation of others typological and structural characteristics of the building influencing the response. This technique considers the influence of some typological features, such as presence of tie rods, number of stories, etc.

In particular, SAVE provides the building classification based on a numerical indicator, SPD, defined as

$$SPD = SPD_V + \sum_{k=1}^{n} q_k + \sum_{i=1}^{m} \sum_{j=1}^{m} \frac{\delta_{ij}(p_i + p_j)c_{ij}}{2m} \tag{3}$$

where SPDv is the classification value evaluated on the basis of the vertical typology only, q and p are weights related to the n-*th* and m-*th* independent and dependent parameters, respectively, the values cij are the non-correlation coefficients, while δij is the Kronecker operator.

The independent parameters considered by the method are.

- presence of tie rods;
- thrusting roof;
- plan regularity;
- infill regularity;
- building position in block;
- reinforced concrete masonry mixed structure;

while the dependent parameters are

- slabs type
- roof structure;
- age of construction;
- number of storeys.

The main steps one should follow to classify a building using SAVE are.

1. Assignment of the most probable vulnerability class on the basis of the EMS-98 classification, considering the vertical structure only;

2. Assignment of the SPDv value on the basis of the vertical classification through given in the first row of Table 1;

3. Evaluation of the vulnerability class for each feature of the building and the corresponding weights p and q;

4. Evaluation of the SPD through Eq. (3);

5. On the basis of SPD, classification of the building according to the SPD ranges given in Table 1.

We refer to Zuccaro and Cacace (2015) for the values of the weights and of the non-correlation coefficients and for further insights into SAVE.

4 Numerical analysis of masonry structures

The analysis of masonry structures is carried out at two scale levels. First, the global-scale behaviour is considered. In doing so, the masonry structure is discretised into bidimensional FE. The structural response is obtained using nonlinear static analysis performed for multiple directions of the seismic action. Then, a local-scale analysis is conducted with the aim of identifying local collapse mechanisms. The result of global and local scale analyses is a relationship between seismic intensities and damage levels.

The adopted numerical framework is implemented in the software POR2000, developed by Newsoft (2021). Some outlines of the FE model and on the numerical methods are given in the following. Further details can be found in the user manual (Newsoft 2021) and in the cited reference.

4.1 Global-scale analysis

4.1.1 Finite element modelling

The global-scale analysis is based on a FE macro-modelling of the building, in which the masonry is described as a homogenised material. The slabs are supposed to be behave rigidly on their plane and only the in-plane resistance of the masonry walls is considered.

The software adopts a hybrid-stress membrane FE, named Flex-6 m (Casciaro 2019a, b), which is an extended version of the Flex-6 FE proposed by Bilotta and Casciaro (Bilotta and Casciaro 2002) specialised for the analysis of masonry walls. The FE is quadrangular and is characterised by six nodes, with two DOFs each, four located at the vertices and two at the mid-side of the edges along the height direction to improve the in-plane flexural

Table 1 Vulnerability by using SAVE classification

Vulnerability class	$SP\,D$			
	A	B	C	D
$SP\,Dv$	5	2	1.7	1.4
max $SP\,D$	–	2.0	1.7	1.4
min $SP\,D$	2	1.7	1.4	0

behaviour. Therefore, the FE has an anisotropic behaviour that guarantees a good balance between accuracy and computational cost.

The FE is mixed, namely the displacement and the stress fields are interpolated independently. Additionally, the FE is isostatic, namely the number of stress parameters is equal to the number of kinematic DOFs minus the number of rigid body motion. This choice gives the best performance in assumed-stress FE, as shown in detail in Bilotta and Casciaro (2002). An important feature of Flex-6 m is that the assumed stress field a-priori satisfies the equilibrium equations for zero bulk loads (Madeo et al. 2014, 2021; Liguori and Madeo 2021). This choice ensures good accuracy, even with a rough discretisation in both the linear–elastic (Madeo et al. 2021) and nonlinear range (Liguori and Madeo 2021).

The nonlinear behaviour of the material follows the strategy proposed by Brasile et al. (2010), developed to capture the essential features of the nonlinear response using a coarse-scale model. The nonlinear behaviour, developed in a context of non-associated plasticity, is based on assuming a set of planes on the FE where frictional response can take place. In addition, tensile and compression limit stress are considered. A simplified damage model is considered. It consists in zeroing the resistance of a masonry wall when the horizontal displacement reaches a ductility limit. Two different values of the ductility limit are considered, depending on which mechanisms is activated between in-plane shear or bending.

4.1.2 Nonlinear analysis

On the basis of a numerical model discretised by Flex-6 m FE, the seismic response of the building is obtained through a nonlinear static analysis. First, a shape of the horizontal forces that model the seismic-induced loads is assumed. Then, the unitary load is scaled by an amplification factor f. The relation between the displacements of the structure, u, and f defines the equilibrium path of the structure that is recovered pointwise using a path-following algorithm. In particular, POR-2000 uses an arc-length technique that has been proven to be a robust solution algorithm, even in presence of limit loads and high nonlinearity in the equilibrium path (Brasile et al. 2010). Within each step, the plasticity problem is solved using a predictor–corrector algorithm.

From the equilibrium path, it is possible to evaluate the capacity curve of the structure, that represents the evolution of the applied load with a reference displacement that is evaluated through an equivalence with the strain energy, see Fig. 3. Along this curve it is useful to identify two points, namely the elastic limit, having displacement u_y, and the ultimate point at displacement u_u.

Fig. 3 Capacity curve of a global-scale analysis and definition of the damage
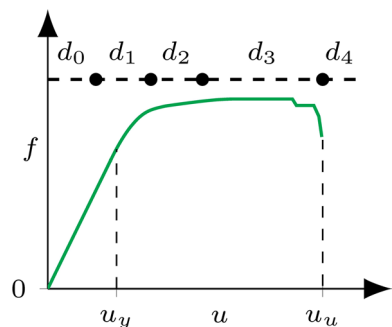
The nonlinear analysis is repeated using two different shapes of the horizontal forces, namely a constant and a linear distribution with the building height. Moreover, the analysis is repeated for 8 different directions of the seismic load. For each structure, the nonlinear analysis is therefore repeated 16 times.

4.2 Local-scale analysis

It is well-known that masonry structures can frequently undergo local collapse. Such circumstance happens, for instance, for old buildings without tie rods and bad connections between walls. In this case, the results provided by the global-scale analysis can be inaccurate. Therefore, for each panel a simplified local-scale analysis is conducted accounting for the most frequent local collapse mechanisms. In particular, simple rocking and vertical flexure are considered in the present work. The analysis, whose details can be found elsewhere (Newsoft 2021; Zuccaro et al. 2017), provides for each structure and for each direction of the seismic load, the value of a_g that causes the activation of the first mechanism, that is defined as a_g^l.

4.2.1 Definition of the damage grades

The evaluations of fragility functions requires the definition of a damage scale. The damage grades are chosen in accordance with EMS-98 (Grünthal 1998), namely.

- d_0: no damage
- d_1: slight
- d_2: moderate
- d_3: extensive
- d_4: near collapse
- d_5: collapse.

When mechanical-based methods are used, a numerical interpretation of EMS-98 damage grades is required. To this end, a heuristic approach is herein adopted. With respect to the global-scale behaviour, the damage grades from d1 to d4 are identified along the capacity curve (see Fig. 3) with the displacements $u_{di}, i = 1 \ldots 4$, defined as (Lagomarsino and Cattari 2014)

$$\bar{u}_{d1} = 0.7u_y, \quad \bar{u}_{d2} = c_2 u_y, \quad \bar{u}_{d3} = c_3 u_u + (1 - c_3)u_y, \quad \bar{u}_{d4} = u_u \qquad (4)$$

where c_2 and c_3 are two coefficients, with c_2 between 1.2 and 2 and c_3 between 0.3 and 0.5. The mechanical model is not capable of capturing the destruction of the building, namely d5.

The seismic action is described by a response spectrum that gives the intensity of the seismic acceleration S_d varying the period of vibration of the structure T. The adopted spectra are regularised and have the following form

$$S_d[T] = \begin{cases} a_g\left(\frac{T}{T_B}F_0 + \left(1 - \frac{T}{T_B}\right)\right) & \text{if} \quad T \leq T_B \\ a_g F_0 & \text{if} \quad T_B < T \leq T_c \\ a_g F_0 \frac{T_C}{T} & \text{if} \quad T_C \leq T_D \\ a_g F_0 \frac{T_C T_D}{T^2} & \text{if} \quad T > T_D \end{cases} \qquad (5)$$

where T_B, T_C, T_D and F_0 are the parameters that control the spectral shape. In order to build fragility curves it is necessary to identify the seismic acceleration a_g that causes each damage grade, defined as $a_{g,i}^g$ for $i = 1 \ldots 4$. To this end, we use the N2 method proposed by Fajfar (1999) that allows to compare the response spectrum with the capacity curve.

Concerning the damage due to local collapse mechanisms, the EMS-98 grades are identified following the hybrid procedure presented by D'Ayala (2005). It has been shown (Zuccaro et al. 2017) how the seismic acceleration that causes a local collapse obtained though a simplified local mechanism analysis corresponds to the d_3 damage grade. Different damage grades are then extrapolated from that value as

$$a_{g,i}^l = \begin{cases} a_{g,i}^{i+1} \frac{i}{i+1} \alpha & \text{if } i \leq 2 \\ a_g^l & \text{if } i = 3 \\ a_{g,i}^{i+1} \frac{i}{i-1} \beta & \text{if } i > 3 \end{cases} \tag{6}$$

where the two coefficients assume the values $\alpha = 0.985$ e $\beta = 1.33$, as suggested by D'Ayala (2005), but can be modified if empirical data or the results of more detailed simulation is available.

Eventually, the value of a_g that causes each damage grade is the minimum between that related to global and local scale behaviour, namely

$$a_{g,i} = \min\left(a_{g,i}^g, a_{g,i}^l\right). \tag{7}$$

The damage d_5 is extrapolated from the other damage levels, as explained in the next section.

5 Mechanical-based fragility curves

The fragility curves are functions of the intensity measure ag giving the probability p that each damage state d_i, $i = 1 \ldots 5$ is reached or exceeded. The fragility curves are well-fitted by a cumulative lognormal distribution (Lagomarsino and Cattari 2014; Zuccaro et al. 2020)

$$p[d \geq d_i] = \Phi\left(\frac{\log\left(\frac{\lambda_i}{a_g}\right)}{\beta_i}\right), \quad i = 1, \ldots, 5 \tag{8}$$

where d is the structural damage, Φ is a normal cumulative probability function, while λ_i and β_i are the mean value and the logarithmic standard deviation of the ag values that cause the di damage grade. For each damage grade d_i, the definition of a fragility curve requires the evaluation of the two parameters λ_i and β_i. To this end, it is necessary to obtain a population of buildings reflecting the in-situ variation of the significant features of the masonry buildings under consideration and to model the uncertainties involved in the seismic vulnerability.

The discrete probability histograms, which give, for a fixed value of a_g, the probability that the structural damage is equal to each i-*th* damage level, are obtained from the fragility functions as

$$\begin{cases} p[d = d_5] = p[d \geq d_5] \\ p[d = d_i] = p[d \geq d_i] - p[d \geq d_{i+1}], \quad i = 4, \ldots, 1 \ . \\ p[d = d_0] = 1 - p[d \geq d_1] \end{cases} \quad (9)$$

Because the numerical method is not capable of giving the value of a_g that produces the damage grade d_5, the relative fragility curve is extrapolated from that evaluated for the damage grade d_4. Therefore, following Lagomarsino and Cattari (2014), the probability that the damage exceeds d_5 is

$$p[d \geq d_5] = 0.8\left(1 - \left(1 - 0.14\,s_d^{1.4}\right)^{0.35}\right)p[d = d_4], \quad s_d = \sum_{i=1}^{4} i\,p[d \geq d_i]. \quad (10)$$

5.1 Uncertainties

One of the main differences between mechanical-based and empirical vulnerability methods resides in the fact that the former require uncertainties to be identified and modelled. In particular, epistemic uncertainties arise from the lack of knowledge of the mechanical and geometrical features of the building under consideration. Such uncertainties can be thereby reduced by increasing the level of detail of field surveys, or by improving the accuracy of the mechanical analyses. Additionally, unavoidable aleatory uncertainties are due to the probabilistic nature of many parameters involved in the risk definition, as loads, measures, material properties. Herein, three uncertainty groups are considered, namely: (i) structural response; (ii) damage level definition; (iii) response spectrum shape.

5.1.1 Uncertainties and variability of the building response

The main purpose of the work is the evaluation of the fragility curves that reflect the specific features of a homogeneous group of buildings. In doing so, it is necessary to obtain the main geometrical and mechanical characteristics that influence the seismic vulnerability. Such information is reflected in the virtual population of structures evaluated using the Monte Carlo generation, as described in in Sect. 2.

5.1.2 Uncertainties on the damage levels

As seen in Sect. 4.2.1, a heuristic interpretation of the EMS-98 damage grade is adopted. However, the damage level determination is subjected to uncertainty. This uncertainty is epistemic, since it is due to the incapability of the numerical model to exactly quantify the damage level of a building according to EMS-98. What happens in real structures is that each damage level can be reached before or after the limits given in Eq. (4) which can be considered only as mean values. The damage levels, affected by uncertainties, are evaluated as

$$\begin{aligned} u_1 &= \bar{u}_1 + rnd(u_y - \bar{u}_1) \quad u_2 = \bar{u}_2 + rnd(\bar{u}_2 - u_y) \\ u_3 &= \bar{u}_3 + rnd(u_c - \bar{u}_3)/2 \quad u_4 = \bar{u}_4 + rnd(u_c - \bar{u}_3)/2 \end{aligned} \quad (11)$$

where *rnd* is a uniformly distributed pseudo-random number to within the range $[-1, 1]$.

5.1.3 Uncertainties on the response spectrum shape

The spectrum shape is subject to uncertainty. Its quantification can be obtained by statistical analysis of seismic records on the area under consideration, or adopting the procedure proposed by Lagomarsino and Cattari (2014).

5.2 The main steps of HAREAS

The main steps of the method are given in the diagram in Fig. 4. The starting point is the survey stage which is divided into two levels. The first one aims at obtaining easily accessible information on all the ne structures under consideration, such as plan dimensions and number of storeys. In particular, the plan shapes are obtained using an automatic algorithm that finds a simple geometry that best fits the actual plan. The second stage focuses on collecting more detailed data through the 2nd level Cartis form. Since Cartis forms are not generally available for all the structures, frequency distributions of the surveyed information are constructed and adopted to describe the variation of the data over the district. The second step of the method consists in generating a sample of n_1 building models using the Monte Carlo method. Each structure is obtained by randomly extracting the data available for all the n_e building and using the frequency distributions evaluated in the previous step. Using SAVE method, a vulnerability class is assigned to this randomly–generated building. Then, the model transferred to POR-2000 which constructs a FE discretisation, performs the nonlinear static analyses for multiple shapes and directions of the seismic action, thereby providing the corresponding capacity curves. The subsequent step of the method consists of a further Monte Carlo analysis that, for each capacity curve, takes into account the variation of the response spectrum and of the damage-state limits. In particular, n_2 random spectra and damage-state limits are generated. Then, using the N2 method, the $a_{g,i}$ values are obtained. Finally, for each vulnerability class, log-normal fragility curves are produced by evaluating λ_i and β_i as mean and standard deviation of the logarithm of the accelerations.

5.3 Validation of the proposed method

The proposed method is tested in order to reproduce analytical fragility curves presented in literature (Lagomarsino and Cattari 2014). The analysed structural typology is identified by the code URM3-M and is characterised by medium height unreinforced masonry buildings having number of storeys uniformly distributed between 3 and 5. Mechanical and geometrical properties vary according to the frequency distributions given by Lagomarsino and Cattari (2014) to which we refer for all the additional details. The in-plane behaviour is only considered.

Figure 5 shows the fragility curves obtained with the proposed Monte Carlo simulation and compared with those proposed by Lagomarsino and Cattari (2014). The proposed method gives higher intensity values for damage grades d_1 and d_2, even if the curves have the same dispersion level. This difference can arise because the structural models are different between the two numerical analyses. Therefore, different models influence the linear solution that is related to low-grade damages. Conversely, in the fragility curves for damages d_3 and d_4 the proposed method gives similar mean intensity
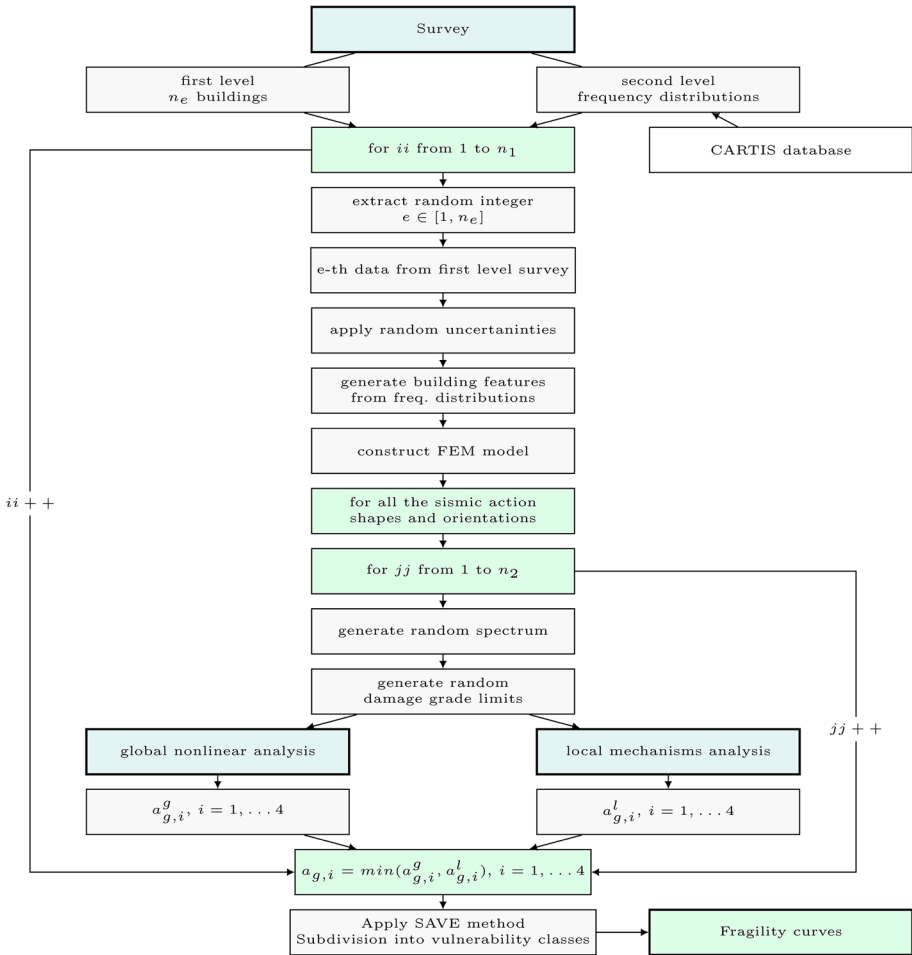
Fig. 4 Diagram showing the main steps of the proposed framework

values and slightly lower dispersion. This can be be due to the different uncertainties propagation methods.

6 Fragility curves for masonry buildings in Cosenza

In this section the proposed HAREAS method is applied to a case study regarding the masonry buildings in Cosenza city centre. First, the random generation of the building population using the Monte Carlo method is presented. Then, the response spectrum affected by uncertainties is evaluated through the analysis of recorded seismic inputs. Finally, the fragility curves are presented and compared with empirical state-of-art curves.

Fig. 5 Fragility curves obtained with the proposed method compared with the results given by Lagomarsino and Cattari (2014)
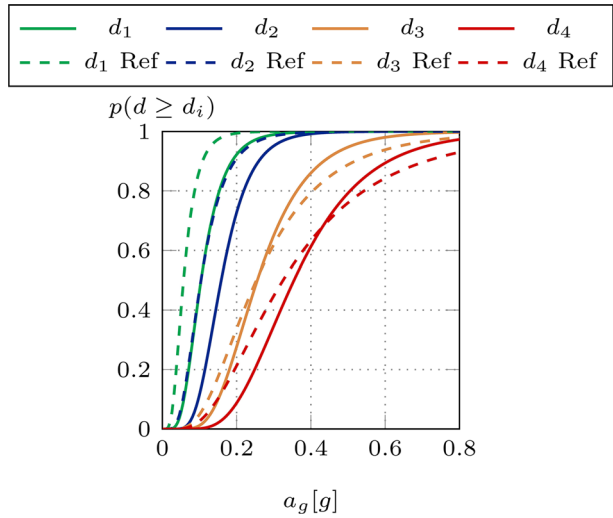
6.1 Exposure model for Cosenza

Mechanical based fragility curves are evaluated for unreinforced masonry buildings in the centre of Cosenza. The buildings under consideration have all been constructed after 1920 and nowadays are surrounded by newer constructions in reinforced concrete. Figure 6 shows a map of the analysed district and highlights the buildings under consideration. The total number of studied structures is 261. Cartis form were collected for the 8% of the buildings under consideration (Fig. 7).
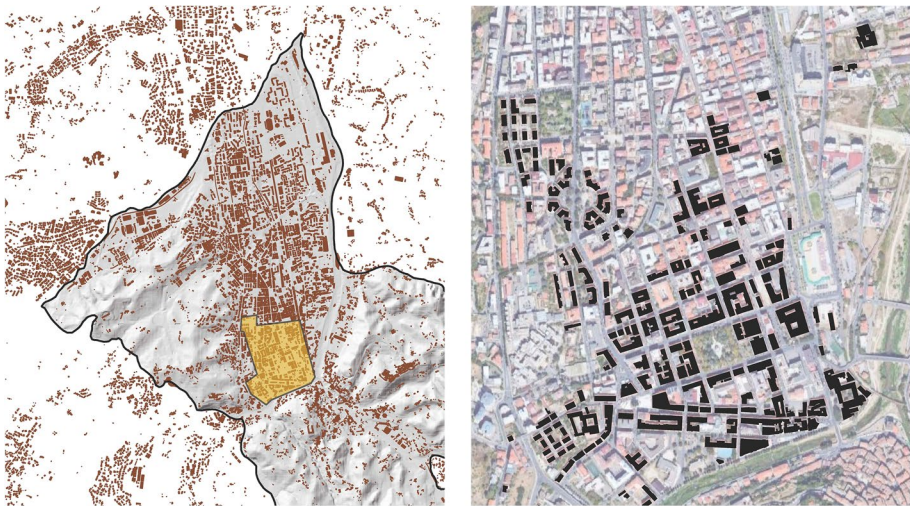
Fig. 6 Location of the studied district on Cosenza urban area (left-hand side) and, in black, the masonry structures considered in the seismic vulnerability assessment (right-hand side)

Fig. 7 Some pictures of the studied masonry structures

The vertical structure is characterised by irregular masonry with horizontal brick lines about every 50 cm. An example of the masonry type is shown in Fig. 8. All the buildings are insulated, but can present plan and elevation irregularity. Since the district under consideration is located on the city centre, the state of maintenance is decent to excellent and the destination is mainly for residential use (Fig. 9).

6.2 Statistic elaboration of survey data

The data obtained in the first-level survey are known for every building, even if random uncertainties are present. Conversely, the data from the second-level survey are known only for a few buildings. In Tables 2 and 3 the results of the second-level survey are given. Other useful information to be used in the Monte Carlo generation are obtained through literature data. For instance, it is imposed that internal walls are not thicker than the external ones, as it is typical for the building under consideration (Campolongo 2009).

The two-levels survey provides deterministic information for some data and frequency distributions for others. On the basis of such information, it is necessary to construct a building population to be analysed with POR-2000. The starting point is the database of buildings given by the first-level survey. Then, the frequency distribution of the structural parameters is assumed to be valid for all the buildings. Some randomly generated models are shown in Fig. 10.

6.2.1 Mechanical properties

The masonry type is only known qualitatively. The Italian code Italian ministry of Infrastructure and Transport (2018) provides ranges of the homogenised mechanical properties
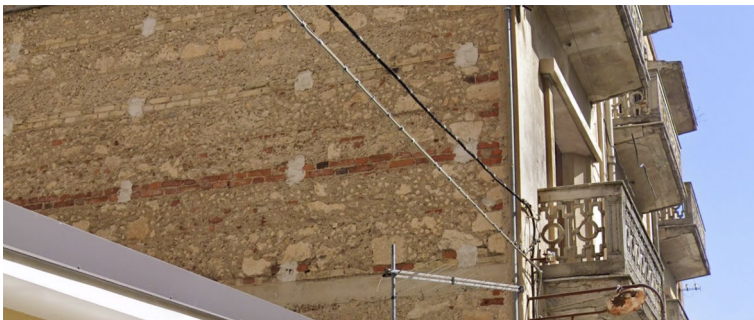

Fig. 8 A picture of the masonry type. It is possible to notice the irregular masonry between horizontal brick lines and the RC slabs

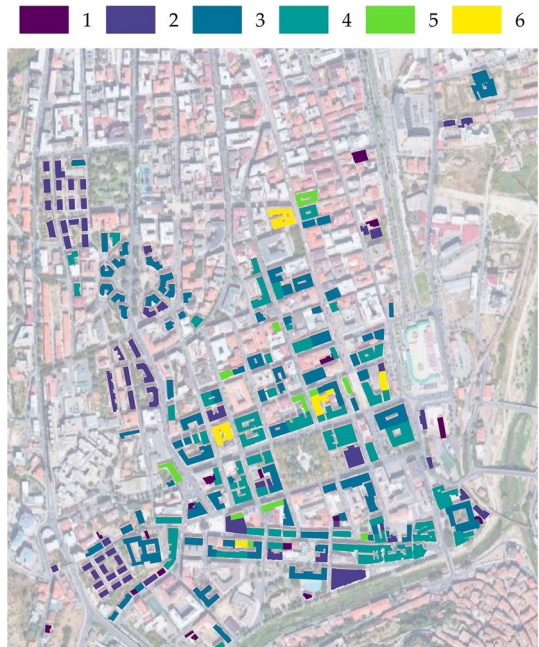**Fig. 9** Number of storeys from the first-level survey

Table 2 Frequency distributions of some building parameters

Property	Mean	Standard deviation	Distribution
Inter-storey height (m)	4.20	0.67	Normal
External walls width (m)	0.75	0.10	Uniform
Internal walls width (m)	0.60	0.10	Normal
walls length (m)	5.00	0.30	Uniform
Facade openings (%)	15	7	Uniform

Table 3 Percentage distribution of some building parameters

Property	Value (%)		
Presence of tie rods	Yes	No	
	80	20	
Slabs	Reinf. concrete	Steel	Wood
	10	70	20
Roof	Reinf. concrete	Steel	Wood
	10	70	20
Masonry type	Irregular masonry		
	100		

of the masonry for a given description of its main features. This information is useful when no in-situ test results are available. Starting from the code ranges, we assume a normal distribution for the mechanical properties, as shown in Table 4.
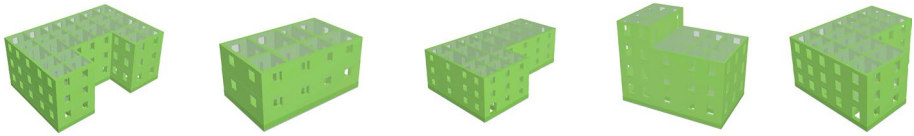
Fig. 10 Some randomly generated building models

Table 4 Mechanical properties of the masonry

Property	Mean	Standard deviation	Distribution
Compression resistance fm (MPa)	4.2	0.78	Normal
Shear resistance $\tau 0$ (MPa)	0.08	0.01	Normal
Elastic normal modulus E (MPa)	1740	240	Normal
Elastic tangential modulus E (MPa)	580	80	Normal
Weight γ (kN/m³)	21	0	–
Ultimate flexural inter-storey drift u^f (%)	0.8	0.1	Normal
Ultimate shear inter-storey drift u^f (%)	0.5	0.2	Normal
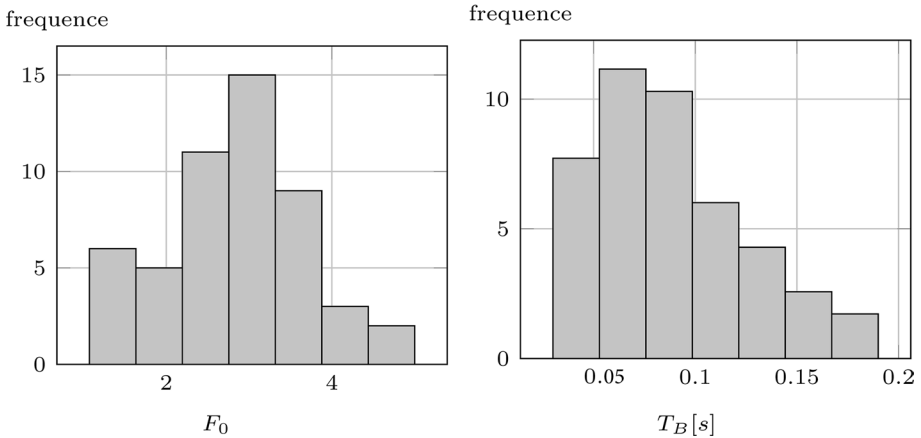
Fig. 11 Frequency distribution for the response spectrum parameters

6.3 Response spectrum affected by uncertainties

As pointed out in Sect. 5.1.3, the earthquake action is described through a response spectrum whose shape is affected by uncertainties. The variability of the spectrum shape is obtained by analysing the seismic inputs registered in a station located in the proximity of the studied district. In particular, the number of accelerograms is 51. For each of them, the response spectrum is constructed considering a damping ratio of 5%. Then, the response spectra are regularised so that they can be described by Eq. (5). The most significant parameters that control the spectral shape are the amplification factor F0 and the period TB at which the constant acceleration part starts. Figure 11 shows the frequency distribution of

F0 and TB which are well fitted by a normal and a lognormal distribution, respectively. In Table 5 the parameters of the response spectrum are reported.

6.4 Fragility curves

In this section we present the fragility curves constructed for the masonry buildings in Cosenza city centre. The Monte Carlo simulation is conducted for a population of $n_b = 10,000$ buildings. For each building, the nonlinear analysis is repeated 16 times, changing the action shape and direction. In the measure of the damage level, we have considered $c_3 = 1.2$ and $c_3 = 0.5$ (Lagomarsino and Cattari 2014). Then, the N2 method is performed, for each capacity curve, $n_2 = 200$ times in order to take into account the uncertainties on the damage grade definitions and on the response spectrum shape. The values of n_b and n_2 have been obtained by studying the convergence of the parameters of the lognormal fragility curves in Eq. (8).

The computational cost required by the proposed approach to evaluate the fragility curves is relatively small. In fact, a single analysis takes, on a Intel i7-11850H 2.50 GHz, an average of 0.08 s, for each seismic load distribution and direction. For a population of 10,000 buildings, on single core, the fragility curves are evaluated in 3.5 h, which can be further reduced by executing the analyses in parallel.

After applying SAVE, the major part of the building are in classes B or C (79%) and only a small fraction in class A (21%). Since the behaviour of class B an C buildings is quite similar, a unique class is considered which is named B–C. Class A buildings are characterised by the absence of tie-rods, high number of levels and irregularity. In such buildings local collapse mechanisms are likely to happen for low values of seismic intensity. Conversely, class B–C buildings have tie-rods or rigid slabs and a global collapse mechanism is expected.

Figure 12 shows the fragility curves obtained for the two vulnerability classes. Additionally, the parameters of the lognormal functions are given in Table 6. Class A fragility curves are characterised by higher dispersions and lower mean values that class B–C. This is due to the local-scale collapse mechanisms that characterise the structural behaviour of class A buildings and produce severe damages for low values of the seismic acceleration. Moreover, the contemporary presence of local and global scale damages increases the dispersion of fragility curves, since two different analysis methods are employed. On the other hand, class B–C buildings reach the damage levels almost exclusively for global-scale behaviour. These buildings provide a better response against the seismic action and are less sensitive to the variation of the parameters.

The proposed mechanical-based fragility curves are compared with empirical curves in Fig. 12. In particular, comparison is made with the fragility curves recently proposed by Zuccaro et al. (2020) who employed SAVE method as vulnerability classifier. A close

Table 5 Parameters of the response spectrum affected by uncertainty

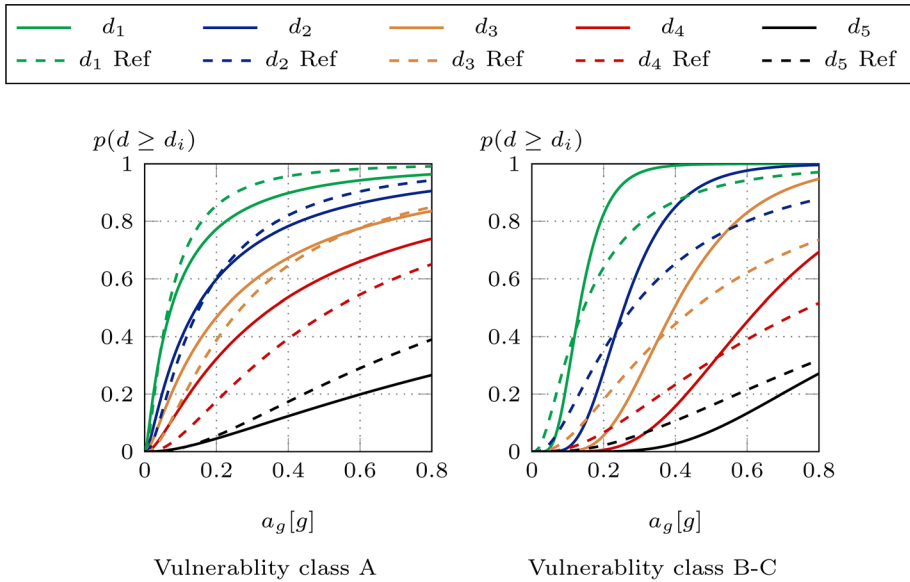| Parameter	Mean	Standard deviation	Distribution
F0	2.84	1.00	Normal
Tb	−2.52	0.44	Lognormal
Td	3	0	–
Tc	Tb/3	0	–

Fig. 12 Fragility curves for the masonry buildings in Cosenza, subdivided for vulnerability classes, compared with the empirical functions proposed by Zuccaro et al. (2020)

Table 6 Parameters of the lognormal fragility functions

Vulnerability class	d_1		d_2		d_3		d_4		d_5	
	$\lambda 1$	$\beta 1$	$\lambda 2$	$\beta 2$	$\lambda 3$	$\beta 3$	$\lambda 4$	$\beta 4$	$\lambda 5$	$\beta 5$
A	0.074	1.324	0.144	1.303	0.223	1.309	0.357	1.257	1.800	1.300
B–C	0.133	0.438	0.257	0.427	0.397	0.432	0.634	0.459	1.050	0.550

correspondence is verified for class A buildings. More distinct differences are observed on class B–C fragility curves, even if a similar trend can be recognised for all the damage levels. Mechanical-based fragility curves are characterised by lower dispersion than empirical curves. This is an expected result. In fact, if empirical curves are constructed on the basis of a heterogeneous building database and are meant to be valid at national scale, the proposed ones are only valid at regional scale. One can say that the differences between the two curves are due to the regionalisation of the analytical results.

The fragility curves of the entire group of masonry buildings are given in Fig. 13. One can observe as they are similar to class B–C fragility curves because class A buildings represent a small fraction of the considered buildings.

Figure 14 shows the probability histograms, evaluated using Eqs. (9) for ag = 0.270 g. This value represents the intensity related to the life-safety limit state according to Italian national code (Italian ministry of Infrastructure and Transport 2018) and is related to a return period of 475 years. Many class A buildings are expected to be seriously damaged, being the d4 the damage grade with the highest probability. The probability of a structural damage is significantly lower for class B–C buildings for which the probability of damage d4 is about 3%. Additionally, it is possible to evaluate a mean damage grade as
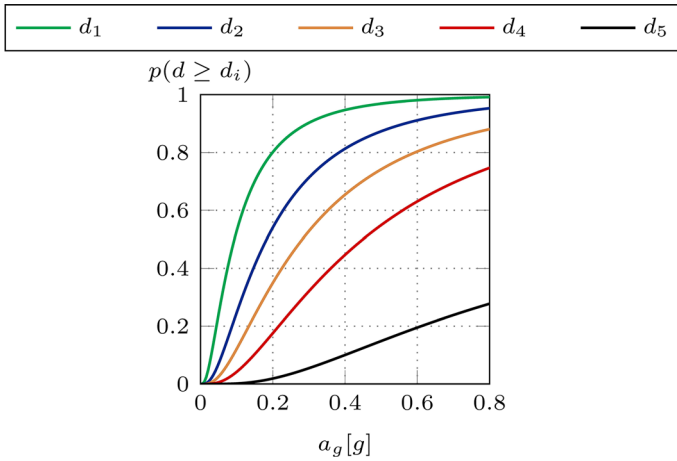
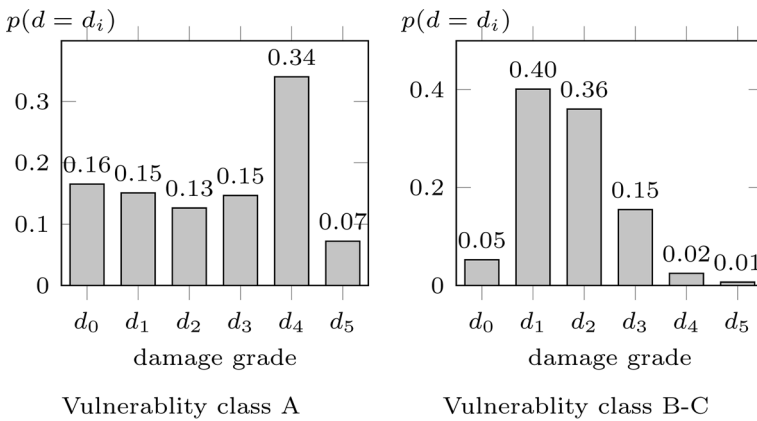**Fig. 13** Fragility curves for the entire building group

Fig. 14 Probability histograms for the vulnerability classes

$$d_m = \sum_{i=0}^{5} i\, p[d = d_i] \tag{12}$$

obtaining $d_m = 2.56$ for class A and $d_m = 1.72$ for class B–C. Interestingly, for medium intensity earthquake the mean damage coincides with the SPD indicator used by SAVE for classes A and B, respectively. If one compares the mean damages with the SPD limits in Table 1, then it can be noticed how the values are in accordance with the building classes.

7 Conclusions

A mechanical-based vulnerability assessment method for unreinforced masonry structures has been proposed. It is based on Monte Carlo simulations to evaluate the propagation of the uncertainties involved in the definition of the seismic risk. Global and local scale collapse mechanisms are considered in the evaluation of the fragility curves. The structural behaviour is obtained using a commercial software, namely POR-2000. It is based on an accurate model of masonry walls using hybrid-stress bidimensional finite elements. Nonlinear static analyses, repeated for multiple directions of the seismic action, are performed to evaluate the building capacity. The analysis is highly efficient, thereby allowing thousands of runs to be executed in little time.

Cartis database is employed to extract data on the buildings, together with a GIS environment. The damage grades are in accordance with EMS-98 definitions and the peak ground acceleration is adopted as intensity measure.

The proposed method is applied to unreinforced masonry buildings located in the city centre of Cosenza. Fragility curves are obtained for two vulnerability classes and are compared with state-of-art empirical results. Two vulnerability classes are identified in the studied area. A more vulnerable one, denoted with "A", characterised by the absence of tie rods or ring beams, is subject to damage states due to local, out-of-plane, mechanisms, while the other, denoted with "B–C", reaches the damage states for in-plane mechanisms. If one considers the extensive damage level, the mean value of the fragility curves goes from 0.357 g of class "A" to 0.634 g of class "B–C". This aspect highlights the high relevance that local collapse mechanisms have on the seismic vulnerability and indicates that buildings in vulnerability class "A" could strongly benefit from retrofit action that limit their occurrence.

The proposed method demonstrates the possibility of performing vulnerability analyses at regional scale and stands as a valid tool that balances accuracy and computational cost. Further extensions of the procedure can regard more complex geometries and masonry buildings in aggregate.

Data availability The datasets generated during and/or analysed during the current study are available from the corresponding author on reasonable request.

Declarations

Competing interests The authors have no relevant financial or non-financial interests to disclose.

References

Abo-El-Ezz A, Nollet M, Nastev M (2013) Seismic fragility assessment of low-rise stone masonry buildings. Earthq Eng Eng Vib 12:87–97

Arkin E, Chew L, Huttenlocher D et al (1991) An efficiently computable metric for comparing polygonal shapes. IEEE Trans Pattern Anal Mach Intell 13:209–216. https://doi.org/10.1145/320176.320190

Asteris P, Chronopoulos M, Chrysostomou C et al (2014) Seismic vulnerability assessment of historical masonry structural systems. Eng Struct 62–63:118–134. https://doi.org/10.1016/j.engstruct.2014.01.031

Baker JW, Cornell CA (2008) Uncertainty propagation in probabilistic seismic loss estimation. Struct Saf 30(3):236–252. https://doi.org/10.1016/j.strusafe.2006.11.003

Benedetti D, Benzoni G, Parisi MA (1988) Seismic vulnerability risk evaluation for old urban nuclei. Earthq Eng Struct Dyn 16:183–201

Bernardini A, Giovinazzi S, Lagomarsino S (2007) Damage probability matrices implicit in scale EMS 98 by housing typologies. Convegno ANIDIS

Bilotta A, Casciaro R (2002) Assumed stress formulation of high order quadrilateral elements with an improved in-plane bending behaviour. Comput Methods Appl Mech Eng 191(15):1523–1540. https://doi.org/10.1016/S0045-7825(01)00334-6

Borzi B, Crowley H, Pinho R (2008) Simplified pushover-based earthquake loss assessment (SP-Bela) method for masonry buildings. Int J Archit Herit 2(4):353–376. https://doi.org/10.1080/15583050701828178

Braga F, Dolce M, Liberatore D (1986) Assessment of the relationship between macroseismic intensity, type of building and damage, based on the recent Italy earthquake data. In: 8th European conference on earthquake engineering

Brasile S, Casciaro R, Formica G (2010) Finite element formulation for nonlinear analysis of masonry walls. Comput Struct 88(3):135–143. https://doi.org/10.1016/j.compstruc.2009.08.006

Buratti N, Minghini F, Ongaretto E et al (2017) Empirical seismic fragility for the precast RC industrial buildings damaged by the 2012 Emilia (Italy) earthquakes. Earthq Eng Struct Dyn 46(14):2317–2335. https://doi.org/10.1002/eqe.2906

Campolongo A (2009) Architettura e metodiche costruttive a Cosenza Nuova. Architecture and construction methods in "Cosenza Nuova" In Italian Gangemi 8849217005

Casciaro R (2019a) Introduction to nonlinear analysis: Flex-m finite element for modelling masonry panels (in Italian). https://doi.org/10.13140/RG.2.2.11623.91049

Casciaro R (2019b) Introduction to nonlinear analysis. part 6: Numerical modelling and analysis of masonry structures (in Italian) https://doi.org/10.13140/RG.2.2.19173.65761

Chieffo N, Formisano A (2019a) Geo-hazard-based approach for the estimation of seismic vulnerability and damage scenarios of the old city of senerchia (Avellino, Italy). Geosciences 9(2):59. https://doi.org/10.3390/geosciences9020059

Chieffo N, Formisano A (2019b) The influence of geo-hazard effects on the physical vulnerability assessment of the built heritage: an application in a district of Naples. Buildings 9(1):26. https://doi.org/10.3390/buildings9010026

Chieffo N, Formisano A (2020) Induced seismic-site effects on the vulnerability assessment of a historical centre in the molise Region of Italy: Analysis method and real behaviour calibration based on 2002 earthquake. Geosciences 10(1):21. https://doi.org/10.3390/geosciences10010021

Chieffo N, Formisano A, Miguel Ferreira T (2021) Damage scenario-based approach and retrofitting strategies for seismic risk mitigation: an application to the historical Centre of Sant'Antimo (Italy). Eur J Environ Civ Eng 25(11):1929–1948. https://doi.org/10.1080/19648189.2019.1596164

Colombi M, Borzi B, Crowley H et al (2008) Deriving vulnerability curves using Italian earthquake data. Bull Eathq Eng 6:485–504

Cosenza E, Manfredi G, Polese M et al (2005) A multilevel approach to the capacity assessment of existing RC buildings. J Earthq Eng 9(1):1–22. https://doi.org/10.1080/13632460509350531

D'Ayala FD (2005) Force and displacement based vulnerability assessment for traditional buildings. SYNER-G: Typology Definition and Fragility Functions for Physical Elements at Seismic Risk. Bull Earthq Eng 3:235–265. https://doi.org/10.1007/s10518-005-1239-x

D'Ayala D, Speranza E (2003) Definition of collapse mechanisms and seismic vulnerability of historic masonry buildings. Earthq Spectra 19(3):479–509. https://doi.org/10.1193/1.1599896

Dolce M, Prota A, Borzi B et al (2021) Seismic risk assessment of residential buildings in Italy. Bull Earthq Eng 19:2999–3032. https://doi.org/10.1007/s10518-020-01009-5

Fadzli Mohamed Nazri CGT, Saruddin SNA (2017) Fragility curves of regular and irregular moment-resisting concrete and steel frames. Int J Civ Eng 16:317–927. https://doi.org/10.1007/s40999-017-0237-0

Fajfar P (1999) Capacity spectrum method based on inelastic demand spectra. Earthq Eng Struct Dyn 28(9):979–993. https://doi.org/10.1002/(SICI)1096-9845(199909)28:9%3c979::AID-EQE850%3e3.0.CO;2-1

Formica G, Sansalone V, Casciaro R (2002) A mixed solution strategy for the nonlinear analysis of brick masonry walls. Comput Methods Appl Mech Eng 191:5847–5876. https://doi.org/10.1016/S0045-7825(02)00501-7

Formisano A, Chieffo N, Clementi F et al (2021) Influence of local site effects on the typological fragility curves for class-oriented masonary buildings in aggregate condition. Open Civ Eng J 15(1):149–164. https://doi.org/10.2174/1874149502115010149

Frankie TM, Gencturk B, Elnashai AS (2013) Simulation-based fragility relationships for unreinforced masonry buildings. J Struct Eng 139(3):400–410. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000648

Furtado A, Rodrigues H, Arede A et al (2016) Simplified macromodel for infill masonry walls considering the out-of-plane behaviour. Earthq Eng Struct Dyn 45(4):507–524. https://doi.org/10.1002/eqe.2663

Grünthal G (1998) European macroseismic scale 1998. Conseil de L'Europe, Luxembourg

Italian ministry of Infrastructure and Transport (2018) Update of the "Technical standards for the constructions". In Italian

Kappos AJ, Panagopoulos G, Panagiotopoulos C et al (2006) A hybrid method for the vulnerability assessment of R/C and URM buildings. Bull Earthq Eng 4(4):391–413. https://doi.org/10.1007/s10518-006-9023-0

Lagomarsino S, Giovinazzi S (2006) Macroseismic and mechanical models for the vulnerability and damage assessment of current buildings. Bull Earthq Eng 4:415–443. https://doi.org/10.1007/s10518-006-9024-z

Lagomarsino S, Penna A, Galasco A et al (2013) Tremuri program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings. Eng Struct 56:1787–1799. https://doi.org/10.1016/j.engstruct.2013.08.002

Lagomarsino S, Cattari S (2014) Fragility functions of masonry buildings. SYNER-G: Typology Definition and Fragility Functions for Physical Elements at Seismic Risk pp 111–157. https://doi.org/10.1007/978-94-007-7872-6

Lamego P, Lourenço PB, Sousa ML et al (2017) Seismic vulnerability and risk analysis of the old building stock at urban scale: application to a neighbourhood in Lisbon. Bull Earthq Eng 15(7):2901–2937. https://doi.org/10.1007/s10518-016-0072-8

Lee TH, Mosalam KM (2005) Seismic demand sensitivity of reinforced concrete shear-wall building using FOSM method. Earthq Eng Struct Dyn 34(14):1719–1736. https://doi.org/10.1002/eqe.506

Leggieri V, Ruggieri S, Zagari G et al (2021) Appraising seismic vulnerability of masonry aggregates through an automated mechanical-typological approach. Autom Constr 132(103):972. https://doi.org/10.1016/j.autcon.2021.103972

Liguori FS, Madeo A (2021) A corotational mixed flat shell finite element for the efficient geometrically nonlinear analysis of laminated composite structures. Int J Numer Methods Eng 122(17):4575–4608. https://doi.org/10.1002/nme.6714

Lourenco P, Rots J (1997) Multisurface interface model for analysis of masonry structures. J Eng Mech 123:660–668. https://doi.org/10.1061/(ASCE)0733-9399(1997)123:7(660)

Madeo A, Casciaro R, Zagari G et al (2014) A mixed isostatic 16 DOF quadrilateral membrane element with drilling rotations, based on airy stresses. Finite Elem Anal Des 89:52–66. https://doi.org/10.1016/j.finel.2014.05.013

Madeo A, Liguori FS, Zucco G et al (2021) An efficient isostatic mixed shell element for coarse mesh solution. Int J Numer Methods Eng 122(1):82–121. https://doi.org/10.1002/nme.6526

Nale M, Chiozzi A, Lambroghini R, et al (2020) Fragility assessment of unreinforced masonry walls undergoing earthquake-induced local failure mechanisms, pp 4311–4317

Nettis A, Gentile R, Raffaele D et al (2021) Cloud capacity spectrum method: accounting for record-to-record variability in fragility analysis using nonlinear static procedures. Soil Dyn Earthq Eng 150(106):829. https://doi.org/10.1016/j.soildyn.2021.106829

Newsoft (2021) Por2000 v.11 user manual. https://docs.newsoft-eng.it/Por-2000

Nikolic Z, Runjic L, Ostojic Skomrlj N et al (2021) Seismic vulnerability assessment of historical masonry buildings in Croatian coastal area. Appl Sci. https://doi.org/10.3390/app11135997

Olivito RS, Porzio S, Scuro C et al (2021) Inventory and monitoring of historical cultural heritage buildings on a territorial scale: a preliminary study of structural health monitoring based on the Cartis approach. Acta IMEKO 10:57–69

Perelli FL, De Gregorio D, Cacace F et al (2019) Empirical vulnerability curves for Italian masonry buildings. COMPDYN 2019 7th international conference on computational methods in structural dynamics and earthquake engineering

Pinto PE (2005) Modeling and propagation of uncertainties. SYNER-G: Typology Definition and Fragility Functions for Physical Elements at Seismic Risk 29–45. https://doi.org/10.1007/s10518-005-1239-x

Polese M, Verderame GM, Mariniello C et al (2008) Vulnerability analysis for gravity load designed RC buildings in Naples – Italy. J Earthq Eng 12(sup2):234–245. https://doi.org/10.1080/13632460802014147

Polese M, Ludovico MD, Porta A et al (2012) Damage-dependent vulnerability curves for existing buildings. Earthq Eng Struct Dyn. https://doi.org/10.1002/eqe.2249

Poljansek K, Casajus Valles A, Marin Ferrer M et al (2019) Recommendations for national risk assessment for disaster risk management in EU. Technical guidance KJ-NA-29557-EN-N (online), KJ-NA-29557-EN-C (print), Luxembourg (Luxembourg), https://doi.org/10.2760/084707 (online), https://doi.org/10.2760/147842 (print)

Rossetto T, Elnashai A (2005) A new analytical procedure for the derivation of displacement-based vulnerability curves for populations of RC structures. Eng Struct 27(3):397–409. https://doi.org/10.1016/j.engstruct.2004.11.002

Rota M, Penna A, Magenes G (2010) A methodology for deriving analytical fragility curves for masonry buildings based on stochastic nonlinear analyses. Eng Struct 32(5):1312–1323. https://doi.org/10.1016/j.engstruct.2010.01.009

Ruggieri S, Cardellicchio A, Leggieri V et al (2021) Machine-learning based vulnerability analysis of existing buildings. Autom Constr 132(103):936. https://doi.org/10.1016/j.autcon.2021.103936

Sandoli A, Calderoni B, Lignola GP et al (2022) Seismic vulnerability assessment of minor Italian urban centres: development of urban fragility curves. Bull Earthq Eng 20(10):5017–5046. https://doi.org/10.1007/s10518-022-01385-0

Sangiorgio V, Uva G, Adam JM (2020) Integrated seismic vulnerability assessment of historical masonry churches including architectural and artistic assets based on macro-element approach. Int J Archit Herit. https://doi.org/10.1080/15583058.2019.1709916

Seyedi DM, Gehl JLNP, Ghavamian S (2010) Development of seismic fragility surfaces for reinforced concrete buildings by means of nonlinear time-history analysis. Earthq Eng Struct Dyn 39:91–108. https://doi.org/10.1002/eqe.939

Simões A, Milosevic J, Meireles H et al (2015) Fragility curves for old masonry building types in Lisbon. Bull Earthq Eng 13:3083–3105

Tocchi G, Polese M, Di Ludovico M et al (2022) Regional based exposure models to account for local building typologies. Bull Earthq Eng 20(1):193–228. https://doi.org/10.1007/s10518-021-01242-6

United Nations (2015) Sendai framework for disaster risk reduction 2015–2030. https://www.preventionweb.net/files/43291_sendaiframeworkfordrren.pdf

Valluzzi MR, Cardani G, Binda L, et al (2004) Seismic vulnerability methods for masonry buildings in historical centres: validation and application for prediction analyses and intervention proposal. 13th world conference on earthquake engineering

Vamvatsikos D, Cornell C (2002) Incremental dynamic analysis. Earthq Eng Struct Dyn 31:491–514. https://doi.org/10.1002/eqe.141

Whitman RV, Reed JW, Hong ST (1973) Earthquake damage probability matrices. In: Proceedings of the fifth world conference on earthquake engineering, Rome, pp 2531–2540

Yang TY, Moehle J, Stojadinovic B et al (2009) Seismic performance evaluation of facilities: methodology and implementation. J Struct Eng 135(10):1146–1154. https://doi.org/10.1061/(ASCE)0733-9445(2009)135:10(1146)

Zuccaro G, Cacace F (2015) Seismic vulnerability assessment based on typological characteristics. The first level procedure save. Soil Dyn Earthq Eng 69:262–269

Zuccaro G, De Gregorio D (2019) Complementary analyses for emergency planning based on seismic risks impact evaluations: Caesar II tool. Conference paper GNGTS

Zuccaro G, Dolce M, De Gregorio D et al (2015) La scheda Cartis per la caratterizzazione tipologico-strutturale dei comparti urbani costituiti da edifici ordinari. valutazione dell'esposizione in analisi di rischio sismico. 34° Convegno Nazionale GNGTS

Zuccaro G, Dato F, Cacace F et al (2017) Seismic collapse mechanisms analyses and masonary structures typologies: a possible correlation. Int J Earthq Eng 2017:121–149

Zuccaro G, Perelli FL, De Gregorio D et al (2020) Empirical vulnerability curves for Italian mansory buildings: evolution of vulnerability model from the DPM to curves as a function of acceleration. Bull Earthq Eng. https://doi.org/10.1007/s10518-020-00954-5