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PREDICTIVE MODEL FOR THE SEISMIC VULNERABILITY ASSESSMENT OF SMALL HISTORIC CENTRES: APPLICATION TO THE INNER ABRUZZI REGION IN ITALY

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ABSTRACT

The paper presents a predictive model for assessing the seismic vulnerability of small historic centres. The model, developed in the framing of other similar methods proposed in the past, needs a limited number of parameters and is based on information collected in the aftermath of the 2009 L'Aquila earthquake.
First, a damage survey carried out on two historic centres hit by L'Aquila earthquake is presented and the most recurrent failure types are classified in terms of severity and extension, leading to damage probability matrices (DPMs). Second, the proposed predictive model is calibrated on the basis of simple observations on the buildings' structural features. Finally, the model is validated through the application to a third historic centre characterized by the same features of the first two case studies. This application proves the generality of the proposed procedure by accurately reproducing the damage that was actually reported after the 2009 earthquake.
The model provides useful information on the most effective anti-seismic strategies that could be implemented at the urban scale for seismic risk reduction.

Keywords: Historic Centres, Seismic Vulnerability Assessment, L'Aquila Earthquake, Damage Scenarios, Masonry Buildings, Risk Mitigation Policies.

1. INTRODUCTION

The seismic activity that has recently rocked the Italian territory has once again highlighted the structural weaknesses of old historic centres, that typically consist of poor masonry buildings that are often characterized by significant fragilities. This statement is particularly true for small historic centres. These are tiny villages, developed in a poor economic contest and without stringent urban regulations, made up of buildings conceived according to a spontaneous "architecture without architects" style (May and Reid, 2010) and erected using rules that local builders applied for satisfying topography and climate needs rather than anti-seismic requirements (Maietti, 2008; De Bernadinis et al, 2008).
Moreover, many historic centres are located in medium-high hazard seismic zones, such as the Alpine and the Apennine chains, with a high exposure, due to the architectural quality and/or historical value of the constructions, as well as to the financial, social and human losses that possible collapses could generate. Thus, according to well known definitions (i.e. Corburn and Spence, 2002; Cardona et al., 2012), the seismic risk is relevant and needs to be mitigated in order to preserve the structural, cultural and functional assets that historical centres host.
The first step in this direction implies setting up reliable predictive tools for the evaluation of the seismic vulnerability of the historic centres' building stocks. These tools must properly account for the intrinsic peculiarities of the historic centres, which are frequently made of clustered buildings whose current aspect is the result of several additions in both plan and elevation, sometimes carried out using and superimposing different materials and local constructive techniques (Da Porto et. al.,

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2013). The global and local structural response of these complex clusters depends on several parameters (Pujades et al., 2012; Senaldi et al., 2012), such as the type of inter-connection between the single structural units, the presence or absence of ring beams, effective iron ties, staggered reinforced concrete slabs, vaulted systems and strengthening interventions that took place over the building life. Moreover, the lack of Building Codes and Regulations has often lead to an irrational expansion of the single building aggregates and of the entire urban layout. As a result, an in depth on site investigation is often necessary to interpret the main construction practices and details used in the historic centres. This in depth survey represents the first and fundamental step for the definition of urban planning strategies for seismic risk mitigation of old historical city centres (Vicente et al., 2015).

Nonetheless, when a vulnerability assessment is carried out at the urban level, a large number of buildings and a large amount of data need to be considered: detailed analyses of the single structures are unpractical and sophisticated models are of scarce interest. Viable vulnerability assessment procedures must be rather simple and can use data from similar buildings hit by past earthquakes. At the urban level, three alternative procedures are typically used for seismic vulnerability assessment (Barbat et al., 2010): *i*) Damage Probability Matrices (*DPMs*), *ii*) Vulnerability Indices (*VIs*), and *iii*) Capacity Curves (*CCs*) based methods.

The above methods usually lead to plot proper fragility curves in a more practical, although more approximate, manner with respect to other numerical and heavy procedures, such as the ones based on the application of sophisticated numerical simulations (Singhal and Kiremidjian, 1996, Kappos, 1995, Barbat et al, 1996), which are usually combined with VI methods (Benedetti and Petrini, 1984).

The DPMs methodology divides the urban area in several building (for example old reinforced concrete buildings, new reinforced concrete buildings, older masonry buildings, new masonry buildings, etc.), grouped according to predefined qualitative descriptors. Each homogeneous group is assigned to a vulnerability class. Vulnerability classes are defined based on damage undergone in past seismic events. For each class, the conditional probability P [D = j | IM] of experiencing a damage level j due to an earthquake of intensity IM is expressed, in a discrete form, as the frequency of buildings that, for that IM, presented that damage level in past earthquakes. An example of such matrices, related to the vulnerability class of steel and reinforced concrete buildings with five or more stories, was proposed by Whitman, Reed and Hong (1973) following to 1971 San Fernando earthquake: nine damage categories, identified by two qualitative damage descriptors and by a damage ratio (damage costs/ building replacement costs), were proposed for five earthquake intensities.

VIs methods are based on the main vulnerability sources for the buildings of a given urban area (building position, lack of box behaviour, thrusting elements, material characteristics, large openings, etc.). A score is assigned to each vulnerability source, measuring its influence on the building structural response. The definition of the vulnerability sources and of their scores is a crucial operation that is carried out by trained experts that must provide consistent judgements during the evaluation process (Maio et al., 2015). All structural data necessary for the definition of the scores are collected during extensive field surveys and are used to fill out a form (for example the GNDT forms, following Benedetti and Petrini, 1984) that yields a vulnerability index i_v. This index is then used to obtain, by means of suitable transformation functions (known as vulnerability functions), a mean damage grade, which, in turn, is related to the conditional probability P [D >j | IM] by proper probabilistic functions (Vicente et al., 2011).

The *CCs* methods broadly identify, during field inspections, the main buildings' geometrical and mechanical features and connect this information to analytical models used for calculating load factors through simplified nonlinear analyses. The determination of the performance points, obtained from push-over curves, leads to predict the damage levels that the structure could experience at different earthquake intensities. Meaningful applications of *CCs* methods were proposed by D'Ayala et al. (2003, 2011), for applications of the FAMIVE procedure, and by

Formisano (2017), Lang and Bachman (2004), Crowley et al. (2004) and several other researches strongly involved in projects dealing with vulnerability assessment at the urban or regional scale, such as the European Risk-EU project (Lagomarsino and Giovinazzi, 2006).
In the above research framework, this paper presents an empirical method for the vulnerability assessment of ancient historic centres and develops it through the application to towns of the inner Abruzzi Region in Italy. The proposed method stems from the studies carried out by the University of Chieti-Pescara for the preparation of the reconstruction plans of fourteen small historic urban centres hit by the 2009 "L'Aquila" earthquake.
The proposed procedure was calibrated on the basis of the observed damage of two meaningful examples, Goriano Sicoli and Poggio Picenze, that experienced different seismic intensities and thus different damage levels. The gathered data helped draw damage scenarios for similar historic centres with the aim of providing a valid support for professionals and decision makers that must plan strengthening actions for reducing the seismic risk at the urban and territorial levels.
First, the paper provide a short description of the 2009 L'Aquila earthquake. Then, it describes the most recurrent failure modes observed in Goriano Sicoli and Poggio Picenze following the 2009 earthquake. Observed damage is classified in terms of severity and extension, leading to specific damage probability matrices (DPMs).
On the basis of the obtained outcomes, a predictive model, which can be used to forecast possible damage scenarios that can be expected in historic centres for earthquakes of increasing intensity, is presented. The model is then validated through its application to another historic centre, Bazzano. Finally, fragility curves for typical Abruzzi historic centres are derived
The proposed model provides a useful tool for identifying the most effective mitigation strategies that could be implemented at the urban scale for effective seismic risk reduction actions.

2. THE 2009 L'AQUILA EARTHQUAKE

The 6.3 M_w shallow earthquake that shook the centre-east part of Italy on April 6th 2009 had the epicentre two kilometres far from L'Aquila, the capital and second most populated city of Abruzzi. It caused 309 deaths, almost 1700 injuries and eighty thousand displaced persons.
The main event was a pure normal faulting mechanism, with a depth of about nine kilometres and a fault length of about 15 km in the SW direction.
The main shock was preceded, during the months before, by a long series of foreshocks that had a maximum peak on the 30th of March, when a M_L 4.1 earthquake was recorded. On the other hand, only four hours before the main shock, a M_L 3.9 event occurred.
Several aftershocks followed in the successive days, culminating in the 5.4 M_w event of April 9th (Chiaraba et al., 2009). Then a long sequence of aftershocks of decreasing magnitude was observed in the following months, as it is shown in Fig. 1, where the earthquake with $M_w \geq 3$ occurred since February to October 2009 are depicted.

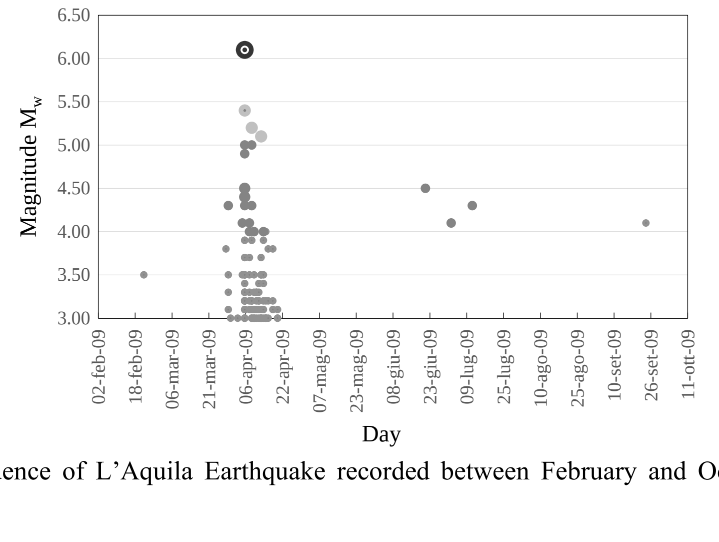


Fig. 1. The seismic sequence of L'Aquila Earthquake recorded between February and October 2009 (Earthquake with $M_w \geq 3$ only).

As for the main event, the record of the station of the Italian Strong Motion Network (RAN) closest to the epicentre (about four kilometres far, on a soil type B), downloaded from the ITACA database (Luzi et al. 2008) and processed according to Paolucci et al., 2011, is shown in Fig. 2a.
At the same manner, in Fig. 2b the corresponding elastic -5% damped- spectrum is given. A maximum ground acceleration of *0.64 g* can be observed and resonance phenomena can be noticed for periods ranging from 0.1 to 0.9 seconds, which represents a domain of interest for the masonry buildings that are studied in this paper.

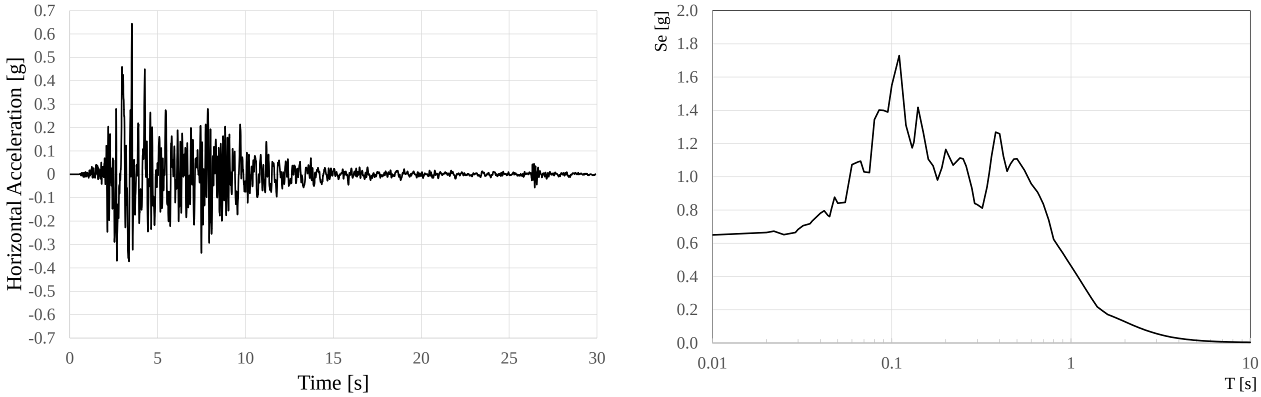


Fig. 2. The record and the corresponding spectrum registered 4 km far from the epicentre of the 2009 L'Aquila Earthquake.

Several researches were carried out in order to investigate the particular features of L'Aquila earthquake. Among these, Bindi et al. (2009) clarified some characteristic effects related to the source, the observed attenuation law and evidenced site effects.
Ameri et al. (2012) studied, through suitable models, the spatial variability of the near-fault strong-ground motions recorded during the mainshock.
Chiaraluce et al. (2011) investigated the geometry of fault segments involved in the 2009 seismic activity by using high resolution foreshock and aftershock locations.
Chioccarelli and Iervolino (2010) highlighted the importance of rupture directivity effects, explaining how some sites, with peculiar positions with respect the epicentre, probably were affected by non-ordinary high amplitude and short duration (impulsive) motions.
Other studies focused on specific sites. Evangelista et al. (2016), for example, focused on the centre of Castelnuovo, concluding that the topographic effects in that centre significantly influenced the ground motion at surface, whereas the role of cavities, which are really spread in Abruzzi centres, seemed to be negligible. Tarque et al. (2015) investigated some soil amplification effects in Poggio Picenze: stochastic analyses were carried out in order to simulate 1,000 different asoil profiles and to run 1,000 simulations, taking into account the inherent variability and uncertainty in the soil profile and on the seismic demand. The same type of analyses were carried out by Di Naccio et al. (2017): the obtained outcomes of this study were referred to the center of Paganica, were the fault mechanism occurred, resulting to be affected by the rupture of the soil.

3. THE TWO HISTORIC CENTRES OF GORIANO SICOLI AND POGGIO PICENZE

3.1 Basis

Goriano Sicoli and Poggio Picenze are located forty-four and twelve kilometres from L'Aquila, respectively. The position of these centres with respect to the epicentre is shown in Fig. 3, where the so-called "cratere sismico" (i.e. the territorial area that was more affected by the earthquake) is evidenced. The two centres have the typical topographic and historical features of the small towns of the Abruzzi mountains (De Matteis et al., 2015). As for the great part of minor historic centres in Abruzzi (Brusaporci, 2007), their current aspect is the result of masonry aggregates of small-medium sizes erected around an urban core characterized by the presence of a church (Tashkov et

a., 2010, Formisano, 2012).
During the 2009 earthquake, Goriano Sicoli and Poggio Picenze experienced a macro-seismic intensity of 7 and 8.5 on the Mercalli-Cancani-Sieberg (MCS) scale, respectively.

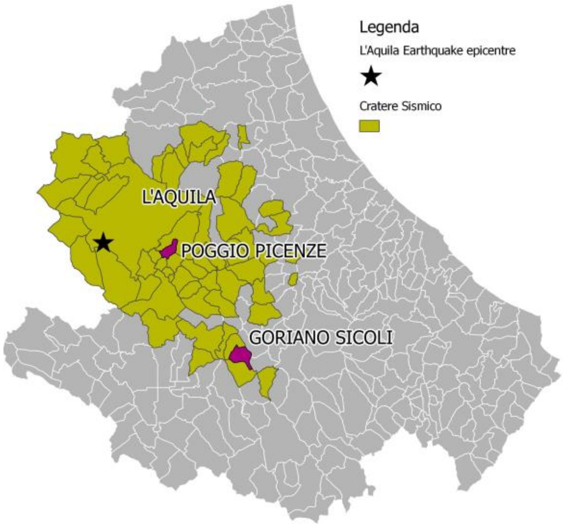


Fig. 3. Map of Abruzzi with the “Cratere Sismico” of the 2009 L’Aquila Earthquake

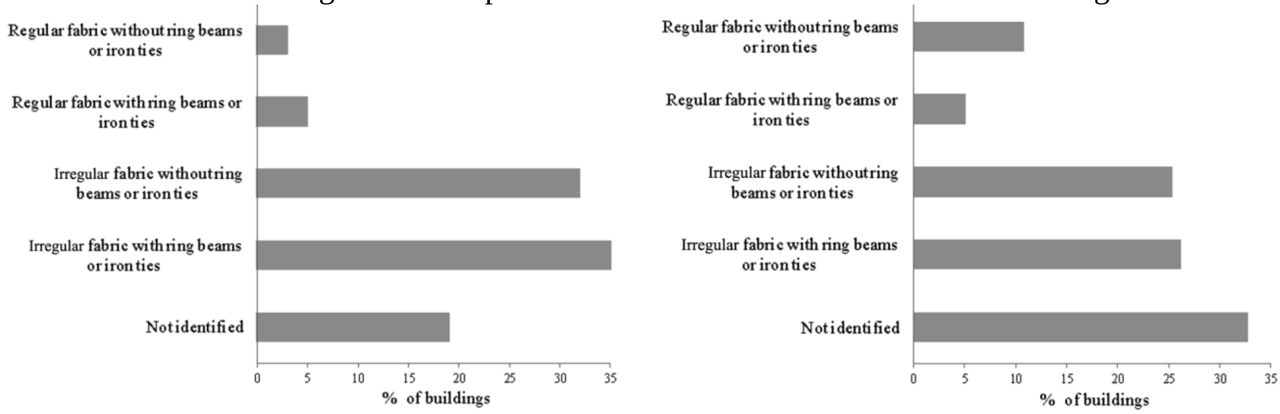
An in-depth survey of the damage caused by the earthquake was carried out in the immediate aftermath of the main shock on more than 350 buildings of the two historic centres, 123 in Goriano Sicoli and 234 in Poggio Picenze. These buildings were selected between the ones characterized by an irregular masonry fabric, they representing, for both the two centres, the majority of buildings.
The survey was done by inspecting each building from the exterior. However, the possibility of observing the internal spaces was often guaranteed thank to the support of local technicians involved in the post-earthquake management phase. Moreover, the building features were also identified by comparing the outcomes of the visual assessment with the contents of the AeDES forms (GNDT, 1986; Bernardini, 2000). On the contrary, these forms were not used for damage evaluation, as the in-field observations highlighted that these were not reliable enough due to the fact that they were filled by technicians who were not adequately trained for the damage reconnaissance.
Each structural unit was identified as part of a cluster characterized by structural systems (often erected at different times) working continuously, with respect to both vertical and lateral loads, from the ground to the roof. Structural discontinuities (change in number of storeys, different in plan layout, misaligned façades, staggered windows, different floors and storey levels, etc.) were carefully reported, as they lead to separate/independent responses.
In a second phase, the reported damage was classified according to the criteria introduced by Grünthal (1998) for the definition of the European Macroseismic scale EMS-98. Six damage levels, D_k, each one associated to a damage score k, ranging from 0 to 5, are defined:

- LevelD_0: No damage;
- Level D_1: Negligible to slight structural damage, with hair-line cracks in very few walls and fall of small pieces of plaster only;
- Level D_2: Slight structural damage and moderate non-structural damage. Cracks in many walls with fall of fairly large pieces of plaster. Partial collapse of chimneys;
- Level D_3: Moderate structural damage and heavy non-structural damage, with large and extensive cracks in most walls; roof tiles detachment; chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls); activation of the first out-of-plane mechanisms;
- Level D_4: Heavy structural damage and very heavy non-structural damage, with serious wall failures; partial structural failure of roofs and floors;

- Level D_5: Very heavy damage to both non-structural and structural parts, with total or near total collapse of the whole building.

3.2 Main features and fragilities of buildings in Goriano Sicoli and Poggio Picenze observed after the 2009 L'Aquila Earthquake

As for the masonry characteristics of the two considered centres, Fig. 4 reports the percentages of the different fabrics. Apart from the masonry that was not identified due to the presence of plaster that prevented any inspection (19% and 32% of buildings for Goriano Sicoli and Poggio Picenze, respectively), irregular fabrics prevail (70% and 52%, respectively). Moreover, in both centres, almost half of the buildings had some protection measures such as ties or effective ring beams.



a) b)

Fig. 4. Masonry fabric distributions in the historic centres of a) Goriano Sicoli and b) Poggio Picenze

Focusing the attention of buildings with irregular fabric, which are of interest for the study carried out in the present paper, these are typically made of three-leaf masonry walls, with rubble limestone rocks with thick, weak layers of lime mortar, characterized by a bonding-to-inert ratio of about 0.5. A chaotic pattern is typical for stones with characteristic sizes smaller than 20 cm: in this case the inclusion of both marl and clay-brick pieces is often observed. On the other hand, a more organized texture can be found for bigger stones which account for about 75% of the wall volume. In both cases, courses are generally absent or, when present, they are not correctly conceived.

These types of masonry layout often recurs in historic centres of inner Abruzzi, as shown Fig. 5 where some representative examples of typical masonry walls in the "Cratere Sismico" are reported. According to the Italian Guidelines "Circolare 617" (2009) these are characterized by a compressive strength in the 1.4 MPa to 2.4 MPa range, a tangential ultimate stress ranging from 0.028 MPa to 0.042 MPa, normal and tangential elastic moduli ranging from 900 MPa to 1260 MPa and from 300 MPa to 420 MPa, respectively.

The above values are in line with the results obtained from in-situ experimental tests carried out by Rovero et al. (2015) in two small villages (Casentino and Sant'Eusanio Forconese) with masonry building characteristics common to all historic centres in Abruzzi. For the mortar quality usually found in dwellings, implemented "double flat jack" tests gave a maximum strength ranging from 1.40 MPa (for irregular stone blocks mixed with rubbles, bricks and roof tiles) and 2.10 MPa (layers of irregular stone blocks interrupted by horizontal brick layers), as well as a normal elastic modulus from 1000 MPa to 1500 MPa.

The masonry layouts described above led, in absence of out-of-plane mechanisms, to a damage that was mainly characterized by diagonal shear cracks, whereas other in-plane failures, such as bed-joint sliding and in-plane bending/rocking, were observed seldom.

Fig. 5. Masonry typologies frequently found in small historic centres of Abruzzi

In Fig. 6, example of buildings with shear cracks of different thicknesses are shown. Specifically, Fig. 6a represents a typical example of building with hair-line cracks and delamination of small pieces of plaster, which corresponds to a damage level D_1, whereas Fig. 6b reports a damage level D_2 with cracks ticker than 1 mm. In Fig. 6c, a building with cracks ticker than 1 cm characterized by significant extensions, with large fails of plaster, is depicted: a damage level D_3 was reached. In this case, the cracks patterns led to the activation of the former out-of-plane mechanisms, as that one of the wall on the left side of the building in Fig. 6c, that, nevertheless, resulted to be characterised by an almost null development.

Diagonal shear cracks were often observed in the walls of the top storey of the buildings, due to the limited amount of axial force. An example is given in Fig. 6d, where a typical crack corresponding to a damage level of the building *D3* is shown.
On the other hand, diagonal shear cracks were also observed in spandrels with lintels (Fig. 6e), whereas in, absence of those, bending failures were noticed (Fig. 6f).

D1 (GorianoSicoli) a) *D2 (GorianoSicoli)* b) *D3 (Poggio Picenze)* c)

D3 (Goriano Sicoli) d) *D3 (Goriano Sicoli)* e) *D3 (Poggio Picenze)*.. e)

Fig. 6. Diagonal shear cracks on masonry walls associated to damage levels of (a) D_1, (b) D_2 and (c) D_3; D_3 damage level observed in (d) walls with low axial forces and (e) spandrels.

It must be observed that cracks developments were accentuated in specific zones of the buildings characterized by higher stiffness. Overall, the irregularity (in thickness, opening positions) of the walls, the presence of staircases interfering with the walls, the lack of effective floor diaphragms, and the position of the structural unit within the building cluster affected the crack and damage patterns. Also, in areas near the epicentre, near fault effects caused strong vertical accelerations that, combined with low compression stresses, induced crack formation in the upper floor walls.
Frequent failures involved total or partial overturning of the façades, due to the walls' out-of-plane high slenderness and low strength/mass ratio. As reported in Fig. 4, iron ties were found for the 35% and 25% of buildings with irregular fabric, for the historic centres of Goriano Sicoli and Poggio Picenze respectively. These percentages are lower of the ones found by other Authors for other historic centres: for example, Indirli et al. (2013) stated that only about 50% of the older masonry buildings in L'Aquila and in the neighbour historic centres present iron ties.
Also, it must be observed that in some of the observed buildings, the ties are not arranged in the building two main directions and at each floor, thus these buildings do not show a box-like behaviour as not all out-of-plane rotations are fully prevented.

In Fig. 7 some of the out-of-plane overturning mechanisms observed in the two historic centres are shown. These have been associated to different damage levels according to the consequences on the whole building. For example, the tilting mechanism in Fig. 7a has been associated to a damage level D_3, as it was activated, but partially developed.
On the contrary, a damage level D_4 has been associated to the fully developed overturning of Fig. 7b. In this case, the ring beam on the top of the wall was not effective in restraining the activation of the mechanism.
Finally, when the out-of-plane triggered other collapses, such as the horizontal floor failure (Fig. 7c), a damage level D_5 has been assumed.

D3 (PoggioPicenze) a) *D4 (GorianoSicoli)* b) *D5 (PoggioPicenze)* c)

Fig. 7. Out-of-Plane overturning in absence of iron ties associated to damage levels of (a) D_3, (b) D_4 and (c) D_5

Also in absence of iron ties, the overturning mechanisms described above were prevented, in some case, because of effective connections with the orthogonal walls. Indeed, as it can be observed in Fig. 7a, most of the building present effective quoins only at the lower storeys, whereas the walls on the top are usually disconnected. Moreover, where present along the whole height of the building, this type of detail was found only at the corners, resulting not effective in restraining other mechanisms, such as the flexural mechanism, with vertical cylindrical plastic hinge, shown in Fig. 8a. In this case, a damage level D_3 has been assigned, because the mechanism was activated but not developed.
On the other hand, effective connections between orthogonal walls resulted ineffective against the "overturning with side wing" mechanism shown in Fig. 8b, which was favoured by the presence of opening close to the corner. In this case, a damage level D_4 was assigned, as, even if the mechanism was not fully developed, some local collapse were found.

D3 (Goriano Sicoli) *D4 (PoggioPicenze)*

Fig. 8. (a) Flexural and (b) overturning with side wing of two facades

Another mechanism that was observed, also in presence of restraining elements (ties or ring beams), is the delamination of the internal and/or external leaves of masonry walls. Through the thickness, the three-leaf walls are generally characterized, for about 1/3 of their transverse dimension, by an internal "sacco" consisting of earth and rubble stones, which separates two external stone and mortar leaves with thickness ranging from 15 cm to 30 cm. The external leaves are not generally connected by through elements and this was the main cause of delamination, in particular when slender external/internal leaves were present in the wall. Generally, this type of mechanism (Fig. 9) has been associated to a damage grade D_4, as it corresponded to the collapse of a limited part of the building.

D4 (PoggioPicenze) *D4 (GorianoSicoli)* *D4 (PoggioPicenze)*

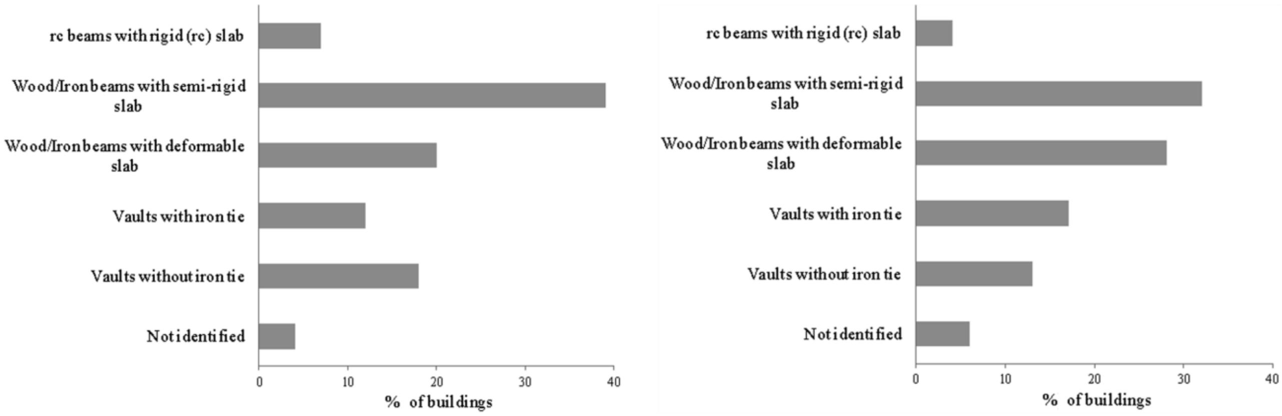
Fig. 9. Delamination of external and/or internal leaves of three-leaf masonry walls

The connections of the walls with the horizontal elements –in particular with the roof system- were another important factor. In most buildings, older timber floors and roofs were unable to restrain the vertical walls' out-of-plane displacements, due to the lack of ring beams, the unidirectional arrangement of the timber beams and the overall in-plane flexibility (Lourenço et al., 2011; Ferreira et al., 2015). In this case, the walls of the upper storeys were prone to out-of-plane mechanisms as they were unrestrained and had little applied axial load. Often, the wall out-of-plane was accompanied by the roof detachment and subsequent collapse, as observed in some cases such as the one that was previously proposed in Fig. 7c.

Giving a glance to the horizontal floors (Fig. 10), one can appraises that they are frequently made of wood beams (20% and 30% in Goriano Sicoli and Poggio Picenze, respectively), whereas almost 10% of the buildings presents reinforced concrete slabs that replaced the original floor system. Brick or thin single-leaf vaults (about 30%) were reported. Although almost half of them strengthened by horizontal ties, these elements were severely damaged during the earthquake, as it is shown in Fig. 11, where damage on both brick and "in folio" vaults is reported.

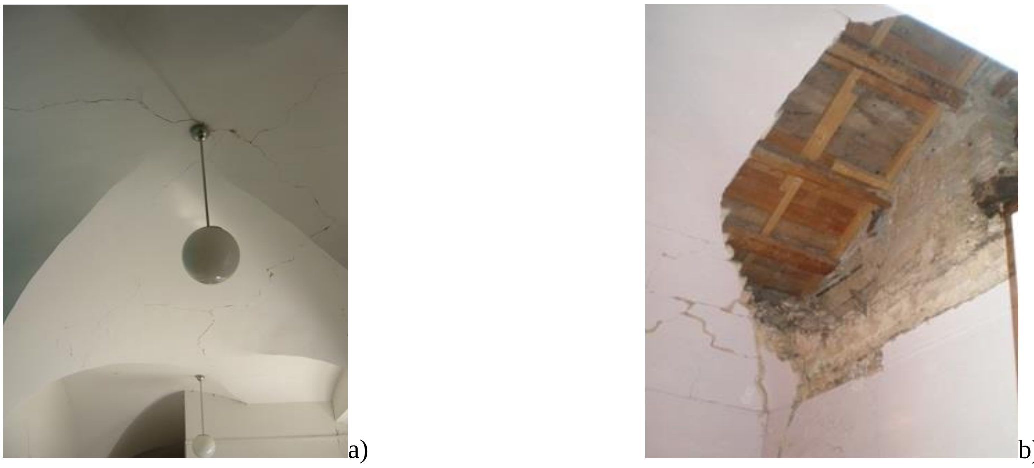
The main roof typologies are reported in Fig. 12. The high percentage of light roofs is due to the fact that many of the original roofs, typically made of wood beams covered by heavy tiles, were recently replaced by steel corrugated sheets.

On the other hand, almost 10% of buildings have heavy roofs that are often the result of recent interventions consisting in the replacement of the existing roofs with heavy reinforced concrete slabs supported by thick reinforced concrete ring beams. This kind of "upgrading" has been recently recognized as detrimental for older masonry buildings, where the new roof translates into a heavy mass at the building top (Criber et al., 2015).



a) b)

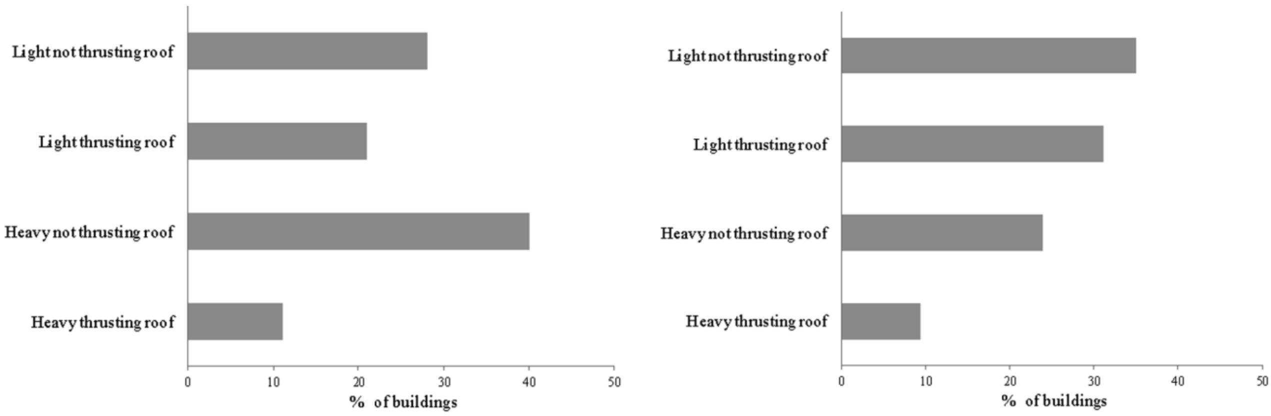
Fig. 10. Horizontal structural elements in the historic centres of a) Goriano Sicoli and b) Poggio Picenze



a) b)

D2 (Goriano Sicoli) *D4 (Poggio Picenze)*

Fig. 11. a) A damaged Brick Vault in Goriano Sicoli and b) a thin "in folio" vault in Poggio Picenze



a) b)

Fig. 12. Roof typologies reported in the historic centres of a) Goriano Sicoli and b) Poggio Picenze

3.3 Damage Provability Matrices of Goriano Sicoli and Poggio Picenze

The frequencies of the damage levels, evaluated according to the criteria shown in the previous Sections, were organized in the Damage Probability Matrices shown in Fig. 13. These matrices allow the prediction, for similar seismic intensities, of likely damage scenarios for historic centres with building features similar to those of the two historic centres considered.

The distributions of the scores *k* associated to each damage level shown in Fig. 13 have mean damage values μ_D= 1.35 and 2.17 for the historic centres of Goriano Sicoli and Poggio Picenze, respectively. These mean values have been obtained according to eq (1)

$$\mu_D = \frac{\sum_{i=1}^{n} D_{k,i}}{n} \tag{1}$$

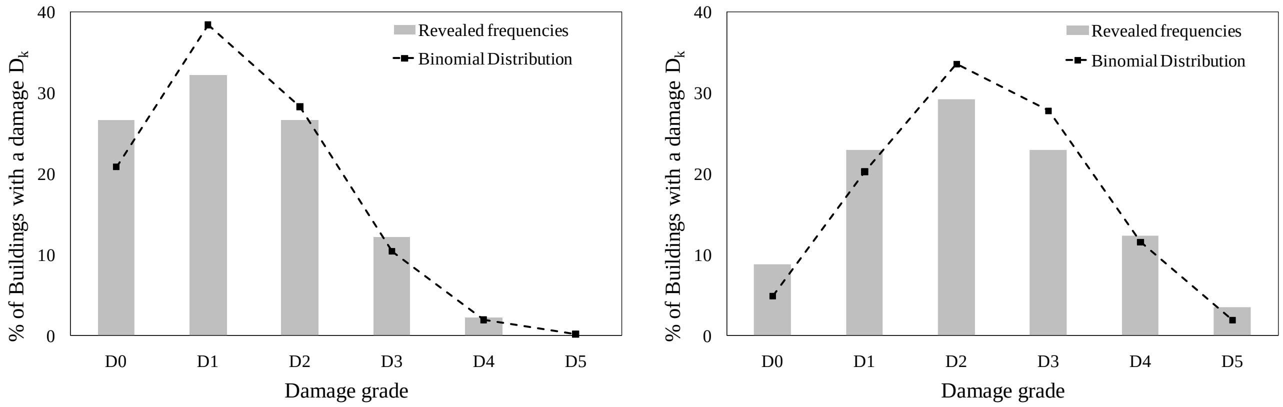
Where *n* is the number of buildings of the analysed stock in the considered centre.
Considering that the two centres have similar features, it is evident that the difference between the two values is mainly due to the different earthquake intensities experienced during the 2009 earthquake, clearly related to the distance from the epicentre. As Chioccarelli and Iervolino (2010) point out, the earthquake was characterized by impulsive features, particularly within a thirty kilometres radius from the epicentre, with strong vertical components and velocity/displacement demands. This input favoured higher damage levels, characterized by the activation/development of out-of-plane mechanisms: several studies have shown that the overturning is correlated to velocity (energy-based parameter) or displacement demands rather than acceleration demands (Doherty, 2000).
Also, it seems that both centres experienced soil amplification effects. For Poggio Picenze, Costanzo and Nunziata (2014), reported soil induced spectral amplifications up to 5–6 for frequencies of 3–4 Hz in the vertical component, and up to 2–3 at 2–6 Hz in the horizontal directions. Most of the masonry buildings in the study area fall in the above frequency ranges: for example, Vestroni (2008) found that, for a typical building near L'Aquila, the main frequencies (for the first three modes) range from 4 to 6 Hz.
As for Goriano Sicoli, no precise information is available regarding the amplification factors. Nevertheless, several studies highlighted that the most recent part of the village lies on limestone bedrock, whereas the historic centre is on talus debris and alluvial fan, therefore more susceptible to soil amplifications (Lanzo et al., 2010). On the other hand, a retrospective analysis of some significant collapses occurred in Goriano Sicoli, carried out by means of refined FEM models (Brando et al., 2015; Criber et al., 2015), evidenced the presence of site effects that induced high vertical accelerations.
Previous studies on the impact of past earthquakes (for example Braga et al. 1982, who studied the 1980 Irpinia, Italy earthquake) found the binomial distribution to be effective in representing the observed damage level distribution.
The binomial distribution has the great advantage of being defined by a single parameter, the mean damage level μ_D. The binomial density function p_k for the damage score *k* is:

$$p_k = \frac{5!}{k!(5-k)!} \cdot \left(\frac{\mu_D}{5}\right)^k \cdot \left(1 - \frac{\mu_D}{5}\right)^{5-k} \tag{2}$$

Its standard deviation, which is relevant to understand whether there is a good correlation between observed and expected results, is:

$$\sigma_d = \sqrt{\mu_D \cdot \left(1 - \frac{\mu_D}{5}\right)} \tag{3}$$

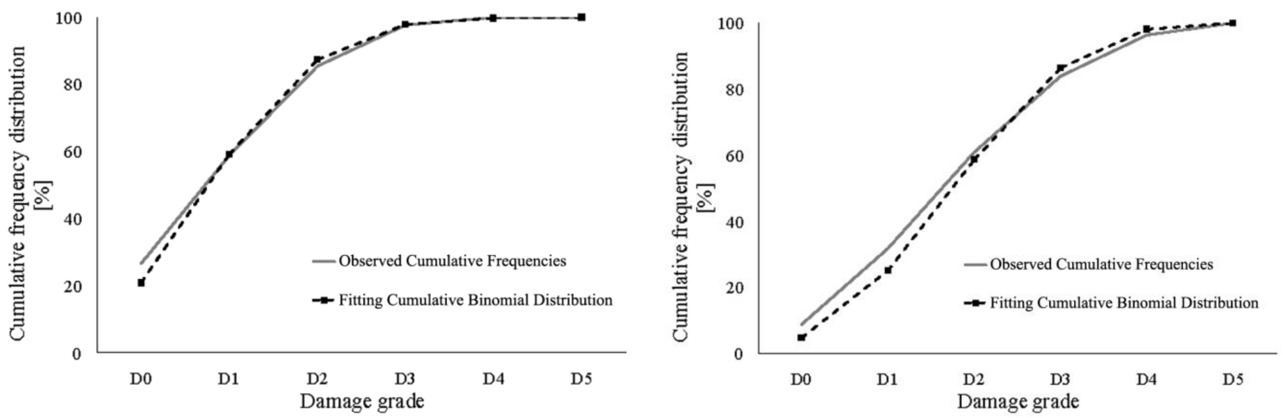
that assumed values of 0.78 and 1.27 for the historic centres of Goriano Sicoli and Poggio Picenze, respectively.
Fig. 13 clearly shows that the binomial distribution-well represents the damage distribution reported in Goriano Sicoli and Poggio Picenze.



a) b)

Fig. 13. Damage Probability Matrices and Binomial Distributions for the historic centres of a) Goriano Sicoli (VII MCS) and b) Poggio Picenze (VIII/IX MCS).

The good agreement between observed and predicted results is even more satisfying if the comparison is carried out in terms of cumulative frequency/distribution function, as shown in Fig. 14. The shown cumulative frequency/distribution functions have been obtained by summing, for each damage level D_k, the frequencies of buildings experiencing a damage level lower or equal to D_k.



a) b)

Fig. 14. Damage Cumulative frequencies vs. Binomial Cumulative Distribution Function for the historic centres of a) Goriano Sicoli (VII MCS) and b) Poggio Picenze (VIII/IX MCS).

4. THE PROPOSED *VI* METHOD AND APPLICATION TO SMALL HISTORIC CENTRES IN INNER ABRUZZI

The previous results indicate that, if a reliable model for assessing the mean damage μ_D for a given earthquake intensity is available, it is then possible to estimate the probability of different expected damage levels D_k by applying the binomial probability distribution of eq (2). Starting from this fundamental assumption, a so-called *VI* method is proposed hereafter, along the same lines of other existing predictive procedures (such as Ferreira et al., 2013) given since the publications of the GNDT forms. The procedure is based on the definition, for each structural unit, of a vulnerability index i_v, expressed as:

$$i_v = \frac{1}{6} \cdot \frac{\sum_{k=1}^{m} \rho_k \cdot (v_{kf} - v_{kp})}{\sum_{k=1}^{m} \rho_k} + 0.5 \tag{4}$$

where

- m is the total number of possible failure sources, also known as vulnerability parameters P_k, that each building can experience.
- ρ_k (ranging from 0 to 1.5) is a coefficient defined in order to weigh the influence that the failure source k has on the global structure stability. 0 indicates that the vulnerability source has no influence on the whole building stability, 1.5 indicates maximum influence.
- v_{kf} is a “fragility indicators”, related to the structural features that contribute to increase the analysed vulnerability.
- v_{kp} is a scores concerning the “protection indicators”, related to anti-seismic devices that, if present, can mitigate the vulnerability for the relative parameter P_k.

The vulnerability parameters P_k are defined based on the buildings typology, as well as on the basis of observations on the effects of past earthquakes on buildings similar to those under consideration. For buildings typical of the historic centres of the Abruzzi mountains, the fourteen vulnerability parameters listed in Tab. 1 were selected based on damage observed after the 2009 earthquake.

Tab. 1. Vulnerability parameters taken into account for the proposed *VI* method

Vulnerability Parameter	Vulnerability type	ρ_k
P_1	Position (in the cluster)	1.5
P_2	Number of storeys	1.5
P_3	1st mode mechanism	1.5
P_4	2nd mode mechanisms	1.0
P_5	Arches	1.0
P_6	Vaults	1.0
P_7	Slabs	1.0
P_8	Thrusting forces	0.8
P_9	Presence of added structures	0.5
P_{10}	Stairs	1.0
P_{11}	Irregularities	0.8
P_{12}	Non Structural elements	0.5
P_{13}	Site effects	1.5
P_{14}	Non Seismic external hazard	0.3

The values of the ρ_k coefficients have been selected according to engineeristic judgements, on the basis of the observed causes of collapse after the 2009 earthquake, but also on the basis of damage reconnaissance activities carried out by other Authors in the past. For example, post-seismic inspections carried out after past earthquakes (among others Neves et al., 2012; Formisano et al., 2017) showed that a building located at the corner of an aggregate is more likely to sustain severe damage (up to *D5*) than a building of analogous features located at the centre of an aggregate block. For this reason, the maximum possible value is assumed for ρ_1. Similarly, it was reported that buildings with a higher number of storeys (typically more than 2), buildings lacking anti-seismic devices resisting out-of-plane (1st mode) mechanisms, or buildings on soils susceptible to site effects are more likely to sustain high damage. Based on these observations, a value ρ_k=1.5 is assigned to the corresponding vulnerability parameters P_2, P_3 and P_{13}.
Even though in plane (2nd mode) failures, related to the poor quality of the masonry fabric or to the lack of specific structural details such as lintels can lead to diffused damage (Damage levels *D3* and *D4*), they do not usually cause extended collapse. Similarly, failures involving vaults, arches, roofs and stairs, are usually associated to local collapses (corresponding to a damage level D4), rather than to collapse of the whole structure. Thus, a value ρ_k=1 is assigned to the parameters P_4, P_5, P_6, P_7 and P_{10}.
ρ_k=0.8 is assigned to parameter P_8, that refers to the presence of thrusting forces. In fact, when 1st mode mechanisms, contemplated by the parameter P_2, are prevented, thrusting forces can cause

deep cracks (Damage levels *D2-D3*), though they are usually limited to localized areas. Similar considerations apply to parameter P_{11}: although irregularities generally lead to cracks and significant concentrated damage in the most stressed parts of the building, global collapse does not typically happen because seismic forces migrate to other, uncracked parts of the building

As for parameters P_9 and P_{12}, a value of ρ_k=0.5 is assigned because they are related to the presence of elements that were added after the initial construction, thus they generally have a partial interaction with the pre-existing structure. On the other hand, the mass increment due to the additional structures is accounted for in parameter P_{11}, which relates to irregularities.

Finally, parameter P_{14} accounts for possible sources of vulnerability from adjacent buildings. For example, the collapse of a building wall may cause damage to adjacent structures. These interferences generally lead to very localised damage, thus a value of ρ_k =0.3 was selected.

As for the scores v_{kf} and v_{kp}, they vary from 0 to 3, thus the vulnerability index of eq. (4) ranges from 0 (no vulnerability) to 1 (maximum vulnerability). Scores v_{kf} and v_{kp} are computed as:

$$v_{kf} = w \cdot z \cdot f \tag{5}$$

$$v_{kp} = w \cdot z \cdot \eta \tag{6}$$

z is a Boolean coefficient indicating the presence/absence (1/0) of the “fragility” of the “protection” indicators; *w* is an “importance factor” whose value can be either 1 or 2. In eq. (5) *w* increases with the number of causes that induce vulnerability of P_k. In eq. (6) *w* is 2 for more effective protection devices, 1 in other cases, as specified later.

The “fragility factor” *f* measures the influence of the causes contributing to the vulnerability indicators related to v_{kf}; it ranges from 0 (if these do not influence the failure activation) to 1.5 (in case of full vulnerability with respect to the onset of failure). Instead, in eq. (6), η (which ranges from 0 to 1.5) allows to judge the effectiveness of the protection devices.

For some parameters, namely P_1, P_2, P_{13}, and P_{14}, the evaluation of *w* and η from eq. (6) is not necessary. In fact, for these parameters *z* is null, as no conventional protection devices is generally present to mitigate the corresponding sources of vulnerability.

The criteria for computing the different factors contributing to v_{kf} *and* v_{kp} are given in

For the present application, the following equation is proposed:

$$V = a + b \cdot i_v^* + c \cdot i_v^{*2} + d \cdot i_v^{*3} = 0.53 + 1.16 \cdot i_v^* - 4.00 \cdot i_v^{*2} + 4.21 \cdot i_v^{*3} \tag{8}$$

Where i^*_v is the mean of the values obtained for each building by applying eq. (4).

In Eq. (8) coefficients *a* (0.53), *b* (1.16), *c* (4.00) and *d* (4.21) are found by imposing the four conditions given in eqs. (9-12).

Tab. 2 and Tab. 3, respectively.

Once the vulnerability indices of the buildings that form the historic centre are evaluated, they are used to define a vulnerability factor *V* that is used to estimate the mean damage as a function of the expected earthquake intensity *I* through the following expression proposed by Sandi et al. (1994):

$$\mu_D = 2.5 \cdot \left[1 + \tanh\left(\frac{I + 6.25 \cdot V - 13.1}{Q}\right)\right] \tag{7}$$

Q is a ductility factor, conventionally assumed equal to 2.3, according to Lagomarsino and Giovinazzi (2006). It is to be considered as a “mean"-”representative” value that accounts for the inelastic capacity of the buildings and that, in more a detailed used of the methodology, could be reviewed and adapted case-by-case since ductility is highly specific and different among the several existing buildings typologies, also with respect to the different masonry layouts shown in Fig. 4.

For the present application, the following equation is proposed:

$$V = a + b \cdot i_v^{*} + c \cdot i_v^{*2} + d \cdot i_v^{*3} = 0.53 + 1.16 \cdot i_v^{*} - 4.00 \cdot i_v^{*2} + 4.21 \cdot i_v^{*3} \tag{8}$$

Where i^{*}_{v} is the mean of the values obtained for each building by applying eq. (4).
In Eq. (8) coefficients *a* (0.53), *b* (1.16), *c* (4.00) and *d* (4.21) are found by imposing the four conditions given in eqs. (9-12).

Tab. 2. Engineering judgement criteria for the evaluation of scores v_{kf}

Parameter	*z* (eq. 4)	*w* (eq. 4)	*f* (eq. 4)
P1 Position (in the cluster)	• 0 when the building coincides with the cluster; • 1 in all other cases.	• 2 for corner buildings in large clusters; • 1 for buildings in the central area of the cluster.	• It is higher as the building is further away from the centre of the cluster. • It is much higher when the aggregate has a rather stretched plan.
P2 Number of storeys	• 0 for one storey buildings; • 1 for more than one storey.	• 2 for more than three storeys. • 1 for buildings with two or three storeys.	• It is related to the ratio between the number of storeys and the footprint of the building, assuming higher values when this ratio grows.
P3 1st mode mechanism	• 1, always.	• 2 when the wall's characteristics are likely to induce overturning (high slenderness, low strength/mass ratio, opening on the transverse walls, no horizontal restraining elements, widely spaced transverse walls, etc.); • 1 when only part of the above possible damage sources are present.	• It takes higher values for low vertical loads, for high slenderness walls, for ineffective floor diaphragm action, when the distance between transverse walls increases, etc.
P4 2nd mode mechanisms	• 1, always.	• 2 for irregular stone blocks mixed with rubbles, bricks and roof tiles; • 1 for layers of irregular stone blocks interrupted by horizontal brick layers.	• It describes, for a given typology, the masonry quality, accounting for the stones' pattern, the mortar-to-inert ratios, the presence of courses, etc.
P5 Arches	• 0 for buildings without arches; • 1 in all other cases.	• 2 for thin arches with the thrust line outside the arch; • 1 for thick arches with the thrust line within the arch.	• It is as higher when the arch/vault configuration-leads to high bending moments for gravity loads. • It is much higher when the supporting elements (piers or walls) are slender. • It must account for the bending moment increment due to seismic forces.
P6 Vaults	• 0 for buildings without vaults; • 1 in other cases;	• 2 for the most vulnerable typologies, such as "in folio vaults; • 1 for other type of vaults.	
P7 Slabs	• 0 for light non-thrusting slabs with well connected beams; • 1 in all other cases;	• 2 for heavy thrusting roofs; • 1 for other slab types.	• It is higher for beams with insufficient cross sections or with scarce vertical wall support. • It strongly depends on the structural health of the slab elements.
P8 Thrusting forces	• 0 for non thrusting elements; • 1 in all other cases;	• 2 for heavily loaded thrusting elements; • 1 for other thrusting elements.	• It is higher for higher ratios between the overturning bending moment induced by the thrusting forces and the out-of-plane strength of the wall
P9 added structures	• 0 for no added structures; • 1 in all other cases;	• 2 for added storey or any major change to the original building layout; • 1 for other added elements, such as balconies, stairs, or for removed elements (i.e. openings).	• Added structures can be judged more or less vulnerable according to their extension, -on or to the interaction level with the original structures.
P10 Stairs	• 0 if stairs are absent; • 1 in all other cases;	• 2 for eccentric stairs with vaulted ramps; • 1 for all other stair types.	• It measures the fragility of the stair construction details
P11 Irregularities	• 0 for regular buildings; • 1 in all other cases;	• 2 for in-plan and in-elevation irregularities; • 1 for in-plan or in-elevation irregularities.	• It accounts for the irregularities' extent and their interaction with the rest of building (for example in terms of staggered floors) • It is higher if a primitive cell is recognizable with respect to the other parts of the buildings
P12 Non Structural elements	• 0 for absent non structural elements; • 1 in all other cases;	• 2 for hollow brick walls, for poor plasters, chimneys, unsafe internal furniture. • 1 when only a part of the above described non structural elements is present.	• It increases, for example, when partition walls are slender, or when the plaster is ineffectively connected to the horizontal and/or vertical structural elements
P13 Site effects	• 0 for bedrock soil and flat terrain; • 1 in all other cases;	• 2 for clay and sloped terrains • 1 for clay or sloped terrain	• It accounts for intermediate situations between rigid subsoil and highly amplifying soils (*f*=1.5); often, this evaluation is supported by seismic microzonation

P14 Non Seismic external hazard	• 0 for isolated buildings without potential interaction with other surrounding constructions; • 1 in all other cases;	• 2 for low distances from surrounding constructions; • 1 for high distances from surrounding constructions;	• It accounts for the damage potentially induced by external elements hitting the building, accounting in particular for the element typology

Tab. 3. Engineering judgement criteria for the evaluation of scores v_{kp}

Parameter	w (eq. 5)	η (eq. 5)
P3 1st mode mechanism	• 2 for buttresses and/or iron ties; • 1 for connections with the transverse walls or reinforced concrete ring beams with transverse diaphragm.	• It measures the effectiveness of the applied protection device. It depends on the number of iron ties, the presence of loose iron ties, insufficient anchor plates, ineffective buttresses, etc.
P4 2nd mode mechanisms	• 2 for reinforced plasters with mortar injections or similar, horizontal and vertical ties in the wall plane, transverse connections within the wall, etc.; • 1 when only part of the above protection strategies are present.	• It is lower when, for example, reinforced plaster is not well connected to the supporting walls (detachment of parts of plaster), or when the tie anchor plates are insufficient.
P5 Arches	• 2 for horizontal ties; • 1 for other reinforcements, in the arch or in the supporting piers.	• It is related to the position of the horizontal steel ties and takes higher values for well strengthened supporting piers.
P6 Vaults	• 2 for horizontal ties, layer of reinforced mortar or carbon fibres sheeting, light filling; • 1 when only a part of the above protection strategies are present.	• It is related to the position of the horizontal steel ties and, in general, takes higher values for all other interventions.
P7 Slabs	• 2 for elements with good connections with the vertical wall and presence of a reinforced concrete screed ; • 1 when elements were added to improve the slab structural performance to gravity loads only (i.e. steel beams, or reinforcement through glued carbon fibres).	• It meausres the effectiveness of the applied reinforcement.
P8 Thrusting forces	• 2 for elements conceived to contrast the thrusting forces; • 1 when the thrusted elements are reinforced.	• It depends on the configuration of the elements conceived to contrast the thrusting force, assuming maximum value when the thrusting force effects are fully counter-balanced. • It accounts for the interventions ŧ on the supporting elements.
P9 added structures	• 2 when there are connecting elements between the original and the added structures, that avoid possible irrregularities; • 1 when an effective connection with the original structures is present, but with irregularities	• It accounts for the connections between the added and the original structures. • For removed elements (i.e. opening) it meaures the effectiveness of the reinstating structures, if present; • It also accounts for the effectiveness of added structures in avoiding irregularities.
P10 Stairs	• 2 for improvements to the stair structural details, and for other structural elements that can reduce the attraction of seismic forces (e.g. walls); • 1 for improvements to the stair structural details only.	• It accounts for the quality of the interventions on the stairs, as well as on the effectiveness of additional elements diverting the seismic forces away from the stairs.
P11 Irregularities	• 2 for added structural elements that can reduce irregularities; • 1 for local interventions interesting the parts of the building that are more stressed die to irregularities.	• It accounts for the effectiveness of the added structures in reducing irregularities (position, typology, etc).
P12 Non Structural elements	• 2 for strengthened non structural elements and for enhanced connections to the masonry walls; • 1 when only one of the above enhancements is present.	• It accounts for the effectiveness of the two enhancements to the non structural elements.

$$\mu_D(I=0;i_v^*=0)=2.5\cdot\left[1+\tanh\left(\frac{6.25\cdot a-13.1}{2.3}\right)\right]=0.001 \tag{9}$$

$$\mu_D(I=11;i_v^*=1)=2.5\cdot\left[1+\tanh\left(\frac{11+6.25\cdot(a+b+c+d)-13.1}{2.3}\right)\right]=4.999 \tag{10}$$

$$\mu_D(I=8.5;i_v^*=0.591)=2.5\cdot\left[1+\tanh\left(\frac{8.5+6.25\cdot(a+0.591\cdot b+0.591^2\cdot c+0.591^3\cdot d)-13.1}{2.3}\right)\right]=2.17 \tag{11}$$

$$\mu_D(I=7;i_v^*=0.68)=2.5\cdot\left[1+\tanh\left(\frac{7+6.25\cdot(a+0.68\cdot b+0.68^2\cdot c+0.68^3\cdot d)-13.1}{2.3}\right)\right]=1.35 \tag{12}$$

Eq. (9) implies that a historic centre with no vulnerability ($i^*_v=0$) presents no damage when it is shaken by an earthquake of minimum intensity I=0.001 (a 0 intensity is not accepted by the model due to the asymptotic nature of the hyperbolic function). On the contrary, according to eq. (10), maximum damage is expected for an earthquake of intensity $I=11$ (that in the Mercalli-Cancani-Sieberg corresponds to the collapse of all masonry buildings) on a historic centre with maximum vulnerability ($i^*_v=1$). Eqs. (11) and (12) reflect the damage scenarios found in the historic centres of Poggio Picenze ($I=8.5$, μ_D=2.17) and Goriano Sicoli ($I=7$, μ_D=1.35), respectively, considering that the average vulnerability indices i^*_v=0.59 and 0.68, respectively, were found following the application of eq. (4) to all analysed buildings. If additional data is available, eq. (8) can be further refined assuming a higher order polynomial.
The shapes that eq. (7) take for Goriano Sicoli and for Poggio Picenze are shown in Fig. 15, where the mean damage is plotted as a function of the seismic intensity. The small difference between the two curves is justified by the presence of a few better performing construction details observed in Poggio Picenze.

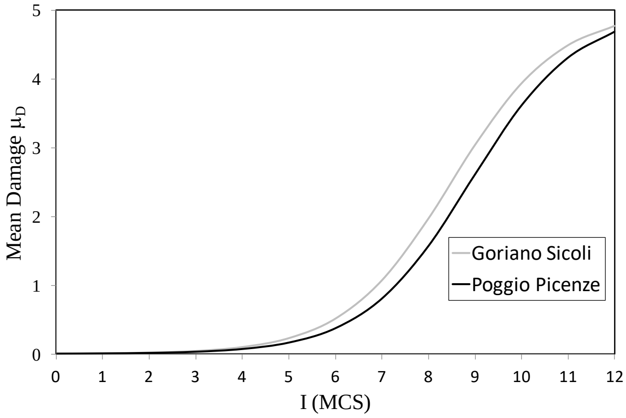


Fig. 15. Mean damage expected for the historic centres of Goriano Sicoli and Poggio Picenze.

For an earthquake intensity equal to 11, the mean damage exceeds 4 (*D4*), implying that all buildings are expected to collapse, consistently with the Mercalli-Cancani-Sieberg scale definition.
The above predictive equations were used to compute the probabilities of attaining the six levels of damage described previously for a historic centre characterized by an average vulnerability index of 0.6. This value was considered for a matter of example, as most of the historic centres that are dealt with in this paper are characterized by a vulnerability index closed to that.
The probability distributions are plotted in Fig. 16.a as a function of the expected macroseismic intensity I_{MCS}.
The corresponding fragility curves are shown in Fig. 16.b: these curves represent the probability of exceeding the first five damage levels considered.
The same results are plotted in Fig. 17 as a function of the expected PGA on rigid soil, obtained by transforming the macroseismic intensity I_{MCS} through the following correlation given by Margottini et al. (1992) on the basis of the observations carried out on the effects of past Italian earthquakes:

$$a_g = c_1 \cdot c_2^{(I-5)} \tag{13}$$

where c_1 is 0.04 and c_2 1.5.
It must be observed that the correlation formula given above presents large uncertainties, as it could be influenced by local site phenomena, type of fault mechanism, directivity issues, etc. For this reason, the curves shown in Fig. 17 are characterized by a certain level of error that, with the support of seismologists, should be deepened, and possible reduce through an adequate knowledge of the site, every time that the proposed method is applied.

The proposed method -and the related fragility curves- appears to be suitable for seismic risk studies and for evaluating possible mitigation interventions at the urban scale in order to reduce the seismic vulnerability of historic centres. The analysis of the distributions of scores v_{kf} and v_{kp} - assigned to the vulnerability parameters for computing the vulnerability indices of the building stocks of the historic centre - helps identify the structural deficiencies with the highest impact on the town vulnerability, clearly indicating where interventions and funding should be directed.

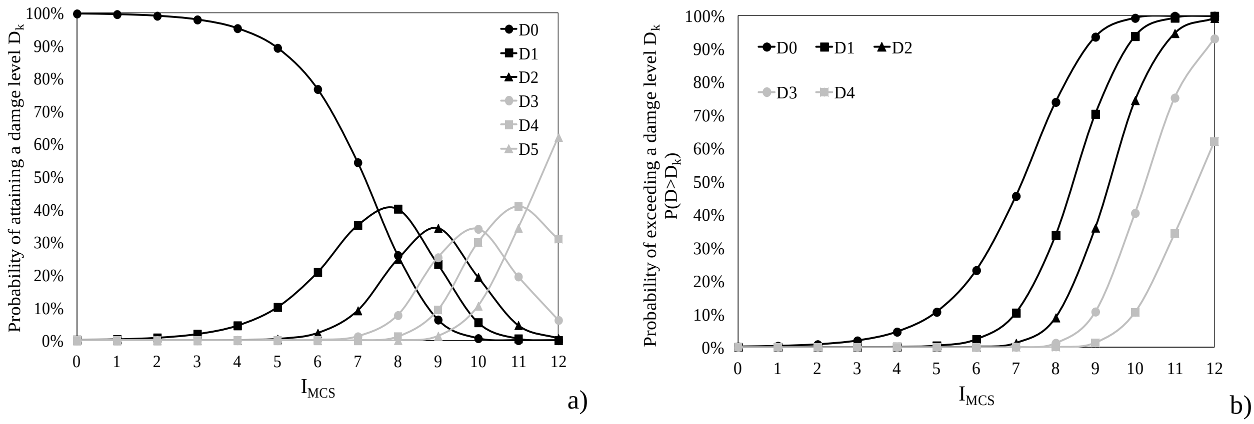


Fig. 16. Expected probability of attaining (a) or exceeding (b) the damage level D_k as a function of the macro-seismic intensity I_{MCS} for a generic historic centre of Abruzzi characterized by an average vulnerability index of 0.6.

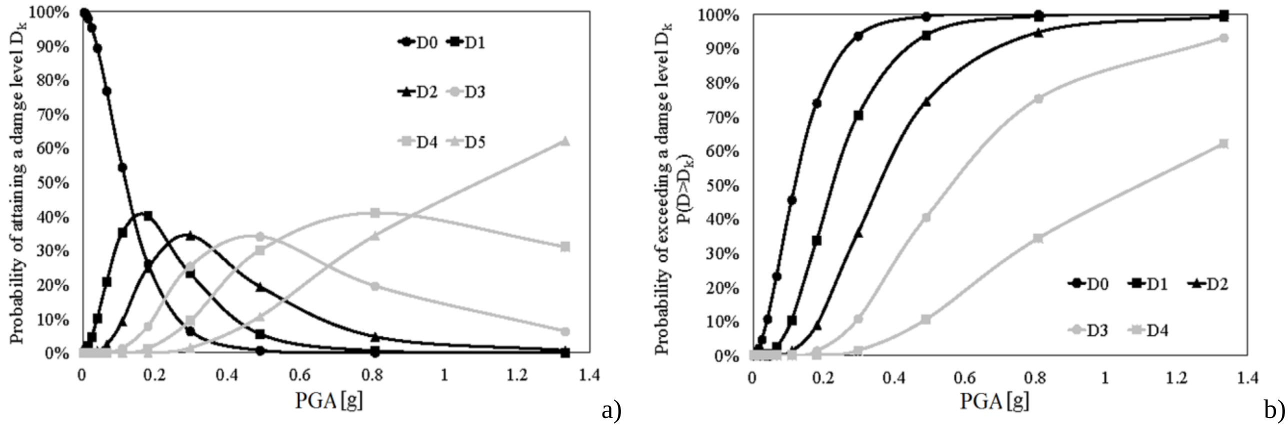


Fig. 17. Expected probability of attaining (a) or exceeding (b) the damage level D_k as a function of the PGA on rigid soil for a generic historic centre of Abruzzi characterized by an average vulnerability index of 0.6.

For example, Fig. 18 shows the values assigned to scores v_{kf} and v_{kp} for the buildings of the historic centre of Poggio Picenze. It is clear that one of the main deficiencies for which retrofitting interventions would be particularly effective is related to the 1^{st} mode mechanisms (parameter *P3*), as well as to the stairs (parameter *P10*), often made of weak vaults without any restraining against thrusting forces.

On the other hand, a significant fragility of the building derives from the position of the structural unit in the cluster (parameter P_1).

Although no specific anti-seismic device is available to mitigate this type of vulnerability (v_{kp}=0), it is apparent that structural interventions aiming at the structural reorganization of the cluster, for example by creating seismic joints, would be particularly effective in reducing the vulnerability of the building stock.

5. VALIDATION OF THE PROPOSED MODEL

The proposed predictive model is based on eq. (7), that is widely used in literature, and, foremost, on the shape that eq. (8) assumes according to the four conditions imposed in eqs. (9-12), two of

which depend on the results from the surveys on the damage of two historic centres following the 2009 L'Aquila earthquake.
If additional data is available (for example from additional historic centres) eq. (8) can be further refined by assuming a higher order polynomial. Nevertheless, pushing the model toward a more precise definition appears to be excessive and unnecessary, in light of the fact that the historic centres of the inner Abruzzi have similar building characteristics in terms of structural layout, material qualities, protection devices, etc.

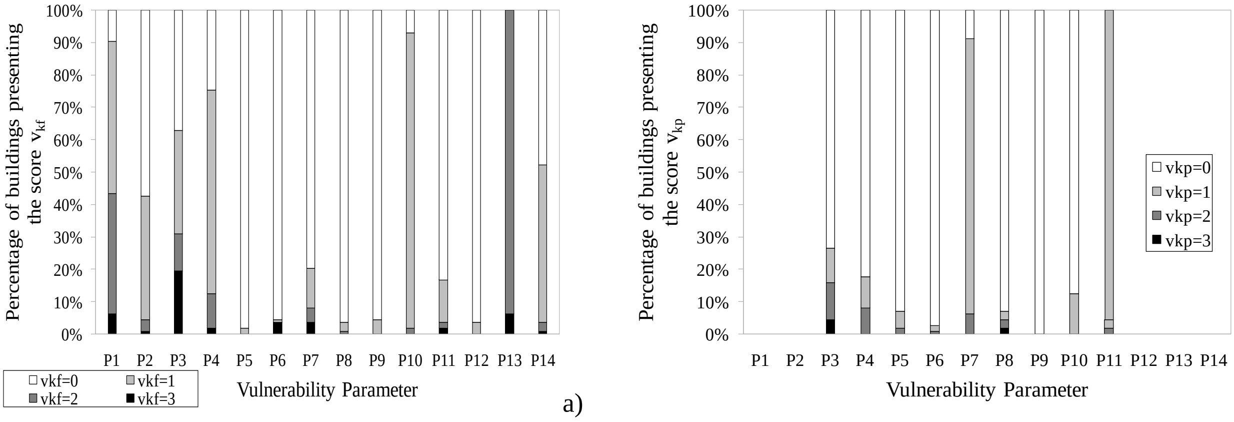


Fig. 18. Distribution of scores v_{kf} (a) and v_{kp} (b) for the buildings of the historic centre of Poggio Picenze

In other words, Goriano Sicoli and Poggio Picenze can be considered representative of other small historic centres in the Abruzzi mountains. Small differences, that are accounted for by the different scores that can be assigned in eqs. (5-6), depend on the specific material and soil characteristics of the different towns.
In order to prove the accuracy of the above general statement and, therefore, the applicability of the damage scenarios obtained from the proposed model to other cases, the model was applied to the study of the historic centre of Bazzano, an independent suburb of L'Aquila. This centre, located 10 km from the epicentre, experienced a macro-seismic intensity of 8 on the Mercalli-Cancani-Sieberg (MCS) scale in the 2009 earthquake. Although total collapses were observed in few buildings only (about 4%), damage grades from *D2* to *D4* were reported. Moreover, the most important building of the centre, the Santa Giusta church, presented partial collapses due to the activation of several mechanisms of the main façades, the lateral walls, the apse and the roof system.
Focusing the attention on ordinary buildings, 124 buildings were analysed. They were grouped in three classes (Fig. 19a). The first class (accounting for approximately 30% of the building stock), contains buildings with pre-2009 interventions (mainly applications of steel ties, ring beams, rigid slabs and, in few instances, by plunging opening). Almost 50% of the buildings have no anti-seismic protection devices. They are representative of a vulnerable building typology often found in the historic centres of the Abruzzi mountains. The last class, that represents approximately 20% of the building stock, was repaired and retrofitted after the 2009 earthquake. This class was ignored in the proposed model application, as it was not possible to retrieve information on the damage provoked by the 2009 earthquake.
Fig. 19b and Fig. 19c report the scores v_{kf} and v_{kp} assigned during the evaluation phase.
These scores highlight a built environment that is different from the two initial case studies, with more significant vulnerability sources. For example, on average, buildings have more storeys, as well as more slender walls. On the other hand, a higher density of effective protection devices, such as iron ties and buttresses, was reported.
The same figure also reports the damage grades (represented by the grey bars) actually recorded for the surveyed building stock after the 2009 earthquake.

The damage grade frequencies computed following the predictive approach described in Section 4 fit quite well the damage grade distribution actually reported after the 2009 earthquake, thus showing the generality of the proposed model in its applicability to other small historic centres of the same region. In fact, according to eq. (4), the above scores yield a vulnerability index i^*_V=0.71 and a mean damage, calculated by eq. (7) for an earthquake intensity of 8 on the MCS scale, of 2.70. For this mean value, the application of eq. (1), with k from 0 to 5 allows to predicts the damage grade distributions represented by the black bars in Fig. 19d.

6. CONCLUSIONS

This paper presented an empirical method, based on the structural damage reported after the 2009 L'Aquila earthquake, for the seismic vulnerability assessment of small historic centre, built on damage data gathered after the earthquake in two representative historic centres of the Abruzzi mountains. The method has been then validated through its application to a third centre, proving to be quite reliable in reproducing the damage scenarios observed after the 2009 earthquake.

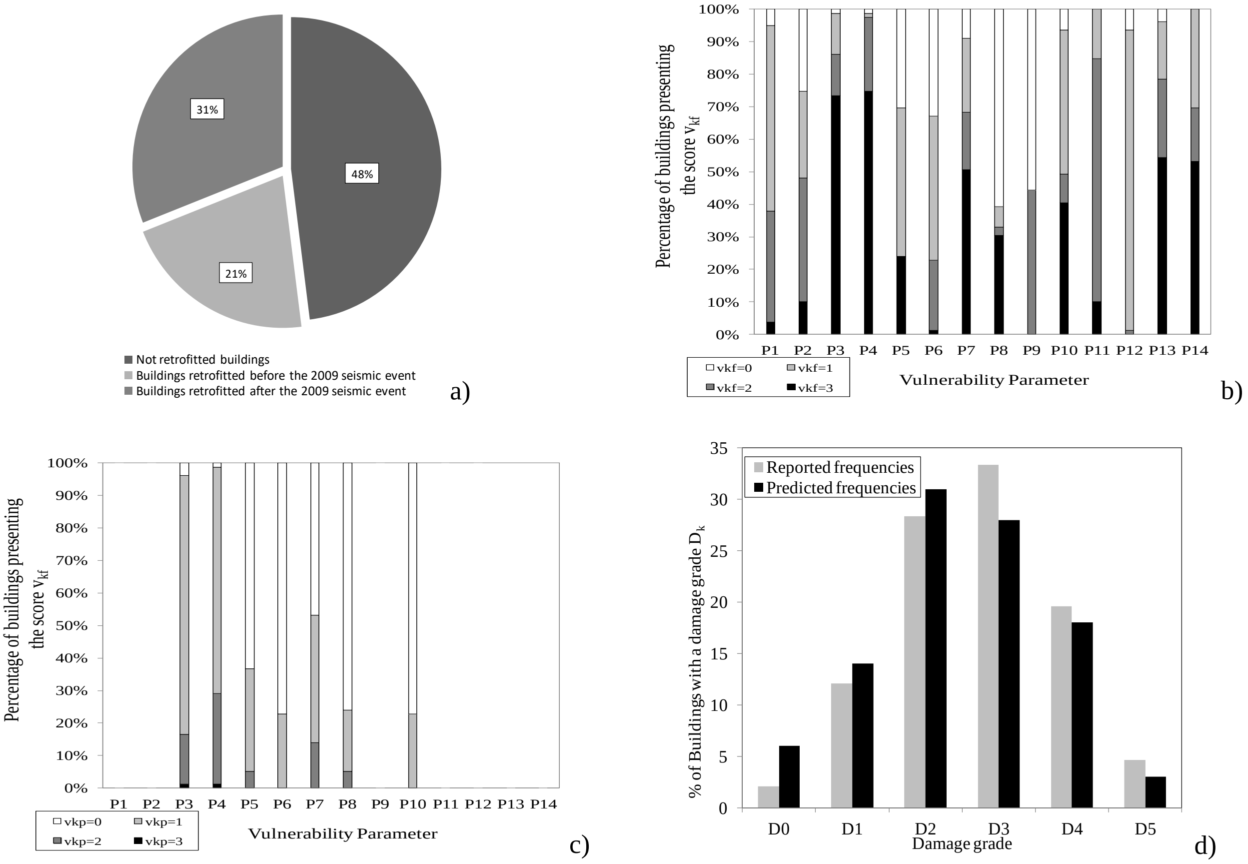


Fig. 19. Validation of the proposed model: a) Building typologies in Bazzano; distribution of the scores v_{kf} (b) and v_{kp} (c) for the buildings of the historic centre of Bazzano; d) reported vs. predicted damage grades frequencies in Bazzano

Starting from the assumption of a binomial damage distribution, the method provides expected mean damage μ_D, which, in turn, is used for assessing the probability of attaining a given damage level for different earthquake intensities.

The method needs the preliminary evaluation of a structural vulnerability index, which is determined on the basis of physical indicators related to fourteen predefined possible sources of vulnerability that can potentially affect the buildings' seismic response. The definition and the distribution of scores for the above indicators help identify the main sources of vulnerability and point out the most suitable mitigation measures to be implemented. The proposed method provides

useful data for the development of specific cost-benefit analyses, that could orient decision making processes for seismic risk mitigation policies in rural areas.
The method recall some of the existing method proposed by other authors in the past, for example in the definition of the vulnerability index, but introduces some novelties, such as those ones related to the definition of the scores -see eqs. (5) and (6)- and to the definition of the vulnerability function, which is given by a polynomial expression calibrated on the basis of the observed damage. The strengths of the approach presented in this paper are listed hereafter:

a) the vulnerability parameters given in Tab. 1 are representative of the historic centres of the inner Abruzzi (and probably of central Italy). They were selected *ex-post* on the basis of in-depth field surveys following the 2009 earthquake. These surveys showed the most common damage mechanisms that are likely to happen if these masonry buildings are hit by different intensity earthquakes (the towns considered are at different distances from the epicentre of the 2009 earthquake and thus experienced different shaking intensities).

b) The new approach proposed for the scores v_{kf} and v_{kp} of eqs. (5) and (6) should grant a less subjective engineering judgement process.

c) The new formulation of the vulnerability factor V in eq. (8) (and the methodology to obtain it) was calibrated on the basis of actually observed damage.

d) The binomial probability distribution is quite accurate in representing the response of the building stock analysed in this study, and can be easily applied to similar situations. Although this result is not entirely new for Italian masonry buildings (see for example Braga, 1982), this paper shows the proposed model accuracy in describing and predicting the response of entire historic centres, starting from a large data set assembled through in-depth inspections on both the exterior and the interior of a very large building stock of masonry structures.

Two final considerations are highlighted:

- even though the proposed method was refined and applied here to building stocks of older, historic masonry buildings, it can be extended to other buildings classes, such as reinforced concrete buildings, through the definition of different physical indicators needed to evaluate the vulnerability index;
- the final goal of these methods is the assessment of damage at the urban scale in a region with homogeneous building characteristics. The results should be integrated to obtain information on the fragility of the different towns at the regional scale, followed by ~~the~~ identification of the main sources of vulnerability that would allow decision makers and stakeholders to optimize private and public investments for the implementation of effective prevention policies and interventions.

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