



Evolution of the seismic vulnerability of masonry buildings based on the damage data from L'Aquila 2009 event

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Abstract

The purpose of this study is the analysis of vulnerability trends, with particular emphasis to the evolution of the seismic behaviour of masonry buildings over the years due to the improvements in construction practices and to the enhancement of building materials over the years, also related to the subsequent enactment of seismic prescriptions. To this aim, residential masonry buildings damaged after the 2009 L'Aquila earthquake are considered, coming from the online platform Da.D.O. (*Database di Danno Osservato*, Database of Observed Damage) recently released from the Italian Department of Civil Protection. General features of all the parameters available from the original database are thoroughly analysed, a selection of which is used for vulnerability analysis, namely the period of construction and the design type, the presence of structural interventions, the type of horizontal structure. Vulnerability curves are obtained through an optimization technique, minimizing the deviation between observed and predicted damage. PGA from ShakeMap is used for ground motion characterization. Damage levels defined according to the European Macroseismic Scale are considered, obtained from the observed damage for vertical structures collected during the inspections. Vulnerability curves are firstly obtained as a function of period of construction and horizontal structural types, limited to the irregular layout and bad quality vertical type only, highlighting their clear influence on seismic behaviour. Lastly, the effectiveness of retrofit intervention is evaluated comparing the vulnerability curves for strengthened masonry buildings compared to those not subjected to any retrofit interventions.

Keywords Vulnerability curves · Residential masonry buildings · Irregular layout masonry · AeDES form · Damage data · Type of design · Strengthening interventions

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1 Introduction

Masonry buildings represent a significant proportion of Italian building portfolio. A large part dates back to several decades ago, even before 1900, thus undergone to all environmental consequences through the years related to degradation of materials, limited by feasible maintenance works and/or retrofitting interventions. On the other hand, the general improvements in construction practices and the enhancement of building materials, also related to all code prescription enacted through the years, have produced a continuous refinement in building industry. Thus, the evolution of seismic behaviour of masonry buildings represents a valuable factor strongly affecting vulnerability analysis of large building stock. The 6th of April 2009 L'Aquila earthquake represents an unprecedented opportunity in this regard, since it strongly affected a large area of Abruzzi Region, among which the ancient city centre of L'Aquila Municipality, whose oldest buildings date back even to the XVIII–XIX century.

Several studies have focused on fragility and vulnerability assessment of Italian existing buildings through numerical approaches (Del Gaudio et al. 2015, 2016, 2018; Borzi et al. 2008; De Luca et al. 2015, 2018; Donà et al. 2020) or the use of post-earthquake damage data (Braga et al. 1982; Sabetta et al. 1998; Orsini 1999; Lagomarsino and Giovinazzi 2006; Rota et al. 2008; Zuccaro and Cacace 2009; Dolce and Goretti 2015; Del Gaudio et al. 2017, 2019, 2020; Rosti et al. 2018). They have investigated about the influence of several factors, i.e. in Braga et al. (1982) 13 different structural typologies (as a function of the quality/layout of vertical and horizontal structural types) are used to identify three vulnerability classes (from A to C). Rota et al. (2008) introduced several building classes, dividing the initial data of 150,000 Italian buildings as a function of vertical structure (masonry, RC, mixed or semi-rigid), number of storeys, vertical and horizontal structure in the case of masonry buildings, and type of design (seismic or non-seismic) in the case of RC buildings. Moreover, Zuccaro and Cacace (2015) proposed a methodology based on “vulnerability factors”, including, among others, the number of storeys and the period of construction.

Moreover, a large number of studies have precisely dealt with (predicted and observed) damage scenario after the 2009 L'Aquila earthquake. For example, in Bernardini et al. (2010), a model, using the conventional definitions of damage degrees and vulnerability classes according to the EMS98 macroseismic scale, has been applied to the entire area of the Abruzzi Region (Central Italy), by means of census inventory of the National Statistics Agency. Some comparisons with observed damage scenario suffered in the town of L'Aquila and surrounding countryside after the 2009 earthquake are also presented. Sabetta et al. (2013) proposed a comparison of the observed number of unusable and collapsed buildings evaluated from damage scenario obtained with the SP-BELA approach (Borzi et al. 2008) and the corresponding predictions based on the SIGE—used by the Italian Department of Civil Protection, with satisfactory results. Similarly, also in Fiorini et al. (2012) the SP-BELA procedure (Borzi et al. 2008) has been used to obtain damage scenario for a subset of buildings located in the historic city centre of L'Aquila using single-building data collected by Tertulliani et al. (2011). Nevertheless, all these studies do not consider the influence of design typology and retrofit intervention on observed damage. On the other hand, other studies presented the results of field investigations carried out after the 2009 L'Aquila earthquake (Indirli et al. 2013; Rossetto et al. 2011; Azzaro et al. 2011). All of them highlighted the great influence of retrofit intervention on damage attitude, which resulted sometimes beneficial if a proper design and construction practise is

conducted and other times detrimental. Overall, the authors underlined that the interventions had played a major role to maintain and improve the stability of masonry structures.

Other studies pointed out the influence of retrofit intervention in vulnerability analysis of masonry buildings struck by L'Aquila 2009 earthquake. D'Ayala and Paganoni (2011) analysed a dataset of residential masonry buildings in the historic city centre of L'Aquila and in the towns of Paganica and Onna based on data from a field survey. The authors highlight the key role played by cross-ties, most widespread in seismic prone areas, in improving the box-like behaviour. Then, they carried out single building analyses, applying the FaMIVE procedure (D'Ayala and Speranza 2003) and the simplified spectral-based procedure proposed in D'Ayala (2005), observing good agreement between the predicted and observed prevalent collapse mechanism and extent of damage. Zucconi et al. (2017, 2018) analyses usability outcomes trends for masonry buildings as a function of macroseismic intensities with several parameters, i.e. with the number of storeys, the period of construction among others and also directly with strengthening interventions clearly highlighting their effectiveness. In D'Amato et al. (2020) a seismic risk analysis of masonry buildings is presented, highlighting the fact that the presence of chains/ring beams is effective only in case of good quality masonry, highly reducing the seismic loss curves and the consequent economic loss.

Moreover, other studies also focused on the effectiveness of structural interventions or traditional reinforcements (i.e., timber tie, wrought iron cross tie inserted in a quoin, steel tie with end plate) in the evaluation of vulnerability and fragility of masonry buildings subjected to various seismic events, i.e. Sisti et al. (2018) and Sorrentino et al. (2019) for what concerns Norcia during the Central Italy 2016–2017 seismic sequence, Binda et al. (1999) and Valluzzi (2007) for Umbria-Marche 1997 earthquake, D'Ayala and Speranza (2003) for Offida, Serra dei Conti, Treia and Cagli cities in Marche Region, Ferreira et al. (2017) for the city centre of Horta, Portugal, Bommer et al. (2002) for Turkey. In all cases, the effectiveness of retrofit intervention on vulnerability of masonry buildings was highlighted by authors if they were properly designed and executed.

Lastly, Dolce and Goretti (2015) examine for the first time the influence of design typology in vulnerability of masonry buildings. In fact, the authors deduce that masonry buildings constructed in L'Aquila city and surrounding countryside after 1915 (year of first seismic classification) can be considered as moderate seismically designed, after a detailed analysis and crosscheck of post-earthquake damage data of last 50 years occurred in Italy. Conversely, masonry buildings dated back to before 1915 show a similar vulnerability compared to those of other areas not yet seismically classified (i.e. the municipalities struck by the Irpinia 1980 earthquake).

In this paper, empirical vulnerability curves in terms of PGA will be obtained from a dataset of about 50,000 masonry buildings, collected after 2009 L'Aquila earthquake and extracted from a wider data collection recently released by the Italian Department of Civil Protection (Dolce et al. 2019) through the online Da.D.O. platform. General features of all parameters available from the original database will be thoroughly analysed, namely the period of construction and the design type (with reference to the complex and articulated regulatory framework of seismic prescriptions), the number of storeys, the presence of structural interventions, the presence of tie rods and/or tie beams, the type of horizontal structure. An additional database, accounting for the “negative evidence” of damage, i.e. representing the undamaged sample although subjected to (low) ground shaking, will be also considered, defined by slightly or even not inspected municipalities of Abruzzi Region obtained through the use of ISTAT 2001 census.

Empirical vulnerability analysis will be performed as a function of a limited number of all considered parameters, which selection will be inspired by a fair compromise ensuring the completeness of the analysis and the reliability of considered subsets of data avoiding an excessive subdivision of the original building stock.

Thus, the comparison of vulnerability curves will be used to quantitatively define the evolution due to the improvements in construction practices and to the enhancement of building materials over the years, also related to the subsequent enactment of seismic prescriptions. Then, the effectiveness of retrofit intervention in seismic vulnerability will also be investigated.

2 The DA.D.O. platform and critical review of L'Aquila post-earthquake data

The Italian DPC (*Dipartimento della Protezione Civile*, Department of Civil Protection), with the support of Eucentre, provided an online platform, called Da.D.O., which allows the access to a large database of buildings, collected during the visual inspections done right after the main earthquakes occurred in Italy in the last 50 years. This database, which considers the last 9 seismic events of national relevance occurred in Italy (Friuli 1976; Irpinia 1980; Abruzzo 1984; Umbria-Marche 1997; Pollino 1998; Molise 2002; Emilia 2003; L'Aquila 2009; Emilia 2012), can represent a useful support in the forecasting and mitigation policies against earthquakes.

Generally speaking, soon after the earthquake the DPC manages and carries out, with the support of technicians from different institutions and professional organizations, an in-situ survey campaign of all the buildings sited in the affected areas, in order to define the safety level of each damaged building, considering also the possible occurrence of aftershocks. Thus, after each event, the main information about location and morphological–functional characteristics of the building, considering also information about the observed damage, are collected. Additionally, information about losses (victims, injured, homeless), Macro-seismic intensity values at municipality level and sometimes even for specific location, about the magnitude of the event and the location of the hypocenter are reported.

The quantity and the quality of the information collected after the 9 considered events result very different, essentially due to substantial changes in the different survey forms used during the inspections. Nevertheless, the DPC with the support of Eucentre has spent huge efforts in the homogenization process of all parameters collected through the years in order to make them comparable between all the 9 considered events.

In general, the parameters collected in the platform relative to each seismic event can be grouped in different macro-sections:

- *Building identification* information about the municipality where each building sited and its position;
- *Building description* number of storeys, inter-storey height, storey surface area, periods of construction and retrofit (if any);
- *Building typology* information on vertical and horizontal structures, on the presence of tie rods or tie beams, of isolated columns, on mixed type structures.
- *Damage* the data on damage strongly depends on the survey form used after each seismic event. For example, for Irpinia 1980 the damage database contains (7 + 1) dam-

age levels (including the null damage) while for Abruzzo 1984 the damage levels are (5 + 1), coherently with the European Macroseismic Scale EMS-98 (Grunthal 1998). Starting from the Umbria-Marche 1997 event, the first level AeDES survey form for post-earthquake damage and usability assessment was used (Baggio et al. 2007), considering (3 + 1) damage levels. Moreover, starting from Irpinia 1980, the damage database contains data for three or more structural components and only from Umbria-Marche 1997 also the damage extension is taken into account.

Table 1 shows the number of buildings available for each database, subdivided as a function of structural types. About 80% of the population is constituted by masonry buildings, while only 8% by RC buildings and the remaining 12% by other types (steel, mixed, ...). For what concerns masonry buildings, 40% of the total is represented by Abruzzo 1984 and L'Aquila 2009 databases. On the other hand, about 70% of the RC buildings is represented by Irpinia 1980 and L'Aquila 2009 databases. Note that Emilia 2003 database is constituted by very few buildings (only 0.4% of masonry buildings and no RC buildings, respectively). Moreover, Umbria-Marche 1997 database reports a significant number of masonry buildings (similar to Abruzzo 1984 and L'Aquila 2009 databases), and a limited number of RC buildings (just 0.2% of the total).

2.1 L'Aquila post-earthquake database

In this study, only L'Aquila database will be considered, with reference to residential masonry buildings, which is characterized by high numerousness (see Table 1), but also by a substantiated reliability, i.e. an huge amount of complete sample of data, as highlighted in previous studies (Rosti et al. 2018; Del Gaudio et al. 2019), since it represents a fundamental requirement for fragility analysis, avoiding the propagation of biases (Rossetto et al. 2013).

From an original dataset of 42,122 residential masonry buildings, only 28,967 are located in municipalities subjected to a complete survey. The latter is defined through the use of a completeness ratio, CR, defined as the ratio between the number of inspections contained in the Da.D.O. platform for each municipality and the number of residential buildings determined through census data (ISTAT 2001). Obviously, the lowest is the CR

Table 1 Structural typologies of the (all and residential) buildings reported in Da.D.O. for each available database

Seismic event	Buildings				Residential buildings			
	Masonry	RC	Others	TOT	Masonry	RC	Others	TOT
Friuli 1976	29,641	469	11,742	41,852	29,641	469	11,742	41,852
Irpinia 1980	30,033	3868	4178	38,079	30,033	3868	4178	38,079
Abruzzo 1984	46,763	2092	2962	51,817	46,763	2092	2962	51,817
Umbria-Marche 1997	41,852	50	6623	48,525	34,150	31	5323	39,504
Pollino 1998	14,515	1285	1642	17,442	11,708	1086	1507	14,301
Molise 2002	19,086	2206	2849	24,141	16,485	1713	1842	20,040
Emilia 2003	899	0	112	1011	808	0	95	903
L'Aquila 2009	49,365	12,019	12,665	74,049	42,122	10,370	10,310	62,809
Emilia 2012	17,881	1795	2878	22,554	12,711	982	1949	15,642

the highest is the number of not inspected buildings; conversely when CR approaches to 1 (or in some cases overcomes 1— due to a different estimate of the total number of buildings between census and survey data) it means that all the buildings sited in that municipality have been inspected. In fact, it can occur that in the area near the epicenter a complete (building-by-building) survey was done, whereas in the area farthest from the epicenter the inspections were done only under building owner's request. In this area (farthest from the epicenter) mainly damaged buildings were inspected, systematically neglecting undamaged ones. Then, if these data will be used for vulnerability assessment without further elaborations, biases in fragility curves could be introduced. To this aim, Rossetto et al. (2013) summarizes the possible solutions adopted in literature to overcome this problem. The first solution deals with the removal of all the data regarding to buildings sited in a municipality characterized by a CR value below a predefined threshold. Values of completeness threshold reported in previous studies are of the order of 0.75 (Sabetta et al. 1998), 0.80 (Goretti and Di Pasquale 2004), 0.60 (Rota et al. 2008), 0.91 (Del Gaudio et al. 2020; Rosti et al. 2020a, b). The second solution (Zuconi et al. 2018, 2020; D'Amato et al. 2020) consists of the identification of incomplete subsets and their integration using census data, considering this additional source as characterized by no-damage to any structural components.

In this study, the mixed approach adopted in (Del Gaudio et al. 2020; Rosti et al. 2020a, b) will be used. Firstly, buildings sited in municipalities characterized by a CR value lower than a predefined threshold, assumed herein equal to 0.91, will be discarded. Obviously, the remaining one will be then used to represent the “positive evidence of damage”.

Then, buildings located in the $CR < 0.1$ municipalities (*slightly surveyed*) together with those not-surveyed of Abruzzi region will be used to account for the “negative evidence of damage”. To this aim, their quantification is taken from census data and their damage characterization is conventionally set assuming they resulted undamaged after the earthquake, since post-earthquake inspections took place very rarely or not at all in these areas. It is to be noted that the census date is not coeval to those when the inspections took place (8 years later). Nonetheless its use can be justified by the fact that the number of buildings constructed in the decade 2001–2011 for this area is substantially negligible.

Figure 1 shows the distribution of the CR for the considered database. The only municipalities with CR higher than 0.91 (i.e. municipalities red-marked in the map in Fig. 1a) will be considered in this study, among those contained in the Da.D.O. platform. These data, which amount to 28,967 out of the 49,365 originally considered, represent the “positive evidence of damage”, coming from post-earthquake inspections. However, the survey form of these 28,967 buildings sometimes do not present all the necessary information (structural type, period of construction, observed damage...). Therefore, in the following, 27,778 buildings characterized by a *complete survey* form will be considered (*damaged database*).

In particular, buildings characterized by a number of storeys between two and three amounts to about 84% of the sample, with a slight predominance of 2-storey buildings. This result is not particularly affected by the period of construction; in fact, only a slight increase in 1-storey buildings against 2- and 3-storey buildings is observed over time (see Fig. 2a).

Figure 2c shows that about 57% of the considered buildings was built before 1919, whereas a roughly uniform distribution can be assumed for building constructed thereafter. Figure 2b shows that a substantial percentage (53%) of the most ancient buildings, i.e. built before 1919, is characterized by vault, a great part, respectively 19% and 22%, by flexible slabs and semi-rigid slabs, and only 6% by rigid slabs. A progressive reduction of percentage of buildings with vaults is observed through the years against an increase of stiffer

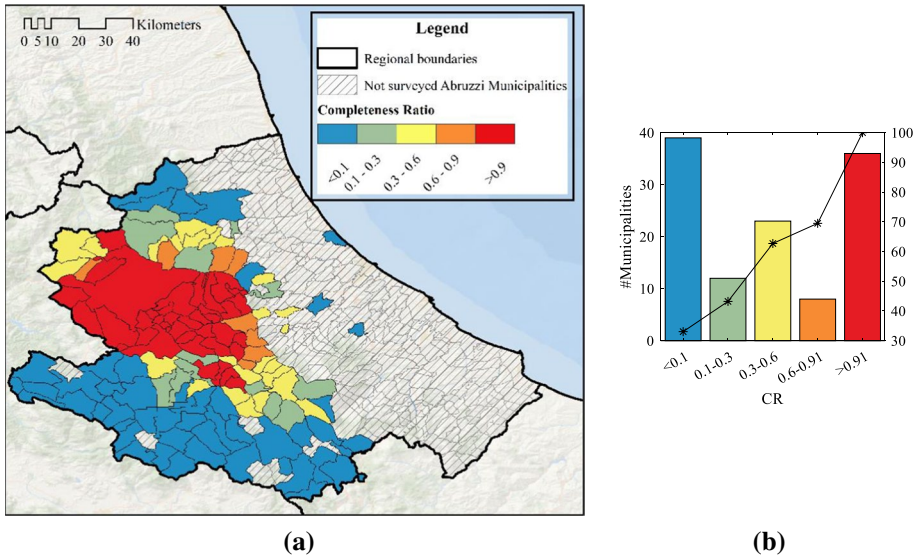


Fig. 1 **a** Spatial distribution of completeness ratio (CR) for Abruzzi Region; **b** CR distribution for all municipalities surveyed after 2009 L'Aquila earthquake

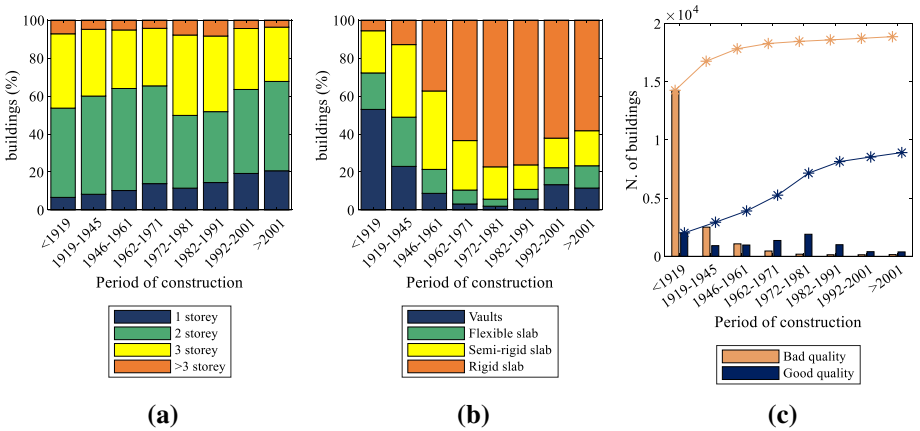


Fig. 2 Period of construction versus **a** number of storeys, **b** horizontal structures and **c** masonry layout distributions for all residential masonry buildings characterized by a complete survey form

horizontal types. In fact, after 1962 the majority (about 70%) of masonry buildings is characterized by rigid slabs.

In Fig. 2c, the distribution of buildings as function of masonry layout and period of construction is shown. It has to be noted that according to AeDES form only bad or good quality outcomes are allowed to characterize masonry layout. Nonetheless, a detailed description on how to assign these outcomes for the most widespread masonry typologies characterizing the Italian buildings stock, supported also by graphic and photographic

documentations, is reported in the AeDES form's Manual. A classification based on two masonry types (*type I* and *II*) is then provided, based firstly on the analysis of external parameters (*first knowledge level*), see Table 2. Clearly, at this level a visual inspection of external or internal masonry layer, thus below the plaster layer, is required to the surveyor. Nevertheless, an in-depth analysis is sometime required for a reliable assignment, especially in case of masonry made of rough-worked element (i.e. *code type B* in Table 2). Thus, additional information are required to reduce these uncertainties: for example, the mortar quality (*second knowledge level*) or the kind of masonry section (*third knowledge level*). However, despite that, uncertain cases characterized by a double classification (*type I/II*) are still provided in Table 2.

It has to be noted that the majority of considered buildings (about 70%) is characterized by an irregular layout and/or a bad quality, defined in AeDES form as masonry *type I*, (i.e. “bad quality masonry”, “BQ” in the following); the remaining 30% of buildings are characterized by a regular layout and/or a good quality, defined in AeDES form as masonry *type II*, (i.e. “good quality masonry”, “GQ” in the following). In particular, only after 1962 the percentage of buildings with regular layout overtakes those of irregular layout. However, buildings dated back after 1962 amount, overall, only to 21% of the total sample. Thus, in the following the focus will be on the BQ masonry buildings, which amount to 18,869 out of 27,778 buildings of considered dataset.

The distribution of number of storeys and period of construction for BQ masonry buildings is then shown in Fig. 3. The comparison between Figs. 2b and 3b shows a clear difference for distribution of horizontal structural type of BQ buildings, due to the removal of post 1962 GQ buildings from the original sample. In fact, after 1962 it can be observed a significant decrease (from about 68% to 32%) in the use of rigid slab for BQ buildings respect to the original database, containing also GQ buildings. An increase of buildings with vault after 1980 is observed for BQ masonry (from about 10% to 35%), probably influenced by indications provided by D.M. 3/3/1975 (see Sect. 3.2).

Moreover, these data are integrated with an additional source of data, standing for the negative evidence of damage, constituted by *undamaged* masonry buildings, located

Table 2 Classification of masonry structures based on material features

Masonry	Type code	Type of elements	Code type	Presence of courses	Masonry type (AeDES)	Masonry type (this study)
Irregular masonry	A	Rounded stones or pebbles	A1	N	I	BQ
				Y	I	BQ
		Raw stones or rubble	A2	N	I	BQ
				Y	I/II	BQ/GQ
Rough masonry	B	Plate-shaped elements	B1	N	I/II	BQ/GQ
				Y	I/II	BQ/GQ
		Pseudo-regular elements	B2	N	I/II	BQ/GQ
				Y	I/II	BQ/GQ
Regular masonry	C	Natural hewn stones	C1	N	I/II	BQ/GQ
				Y	II	GQ
		Artificial stones (bricks)	C2		II	GQ

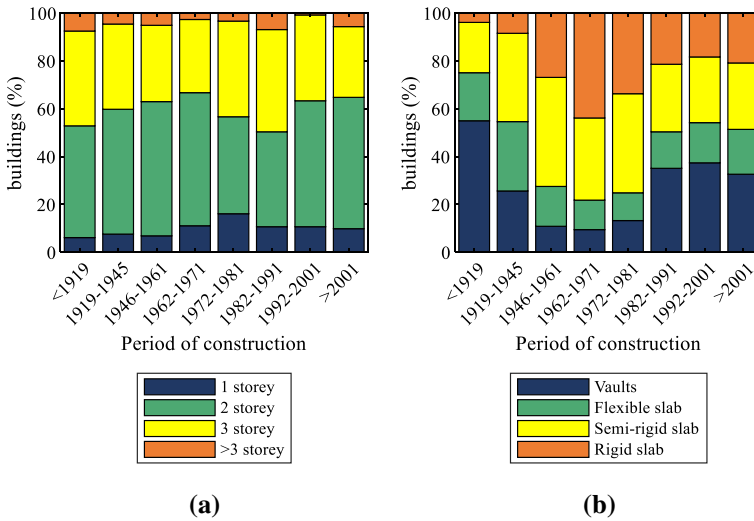


Fig. 3 Period of construction versus **a** number of storeys and **b** horizontal structures percentage distributions for BQ residential masonry building characterized by a complete survey form

in the Abruzzi municipalities subjected to a very small number of inspection ($CR < 0.1$) or not-surveyed after L'Aquila 2009 earthquake (see Fig. 1a), amounting to 143,818 units according to ISTAT 2001 data. Note that this integration allows to reduce potential biases in vulnerability estimation (Del Gaudio et al. 2020; Rosti et al. 2020a, b), due to the un-representativeness of the sample (a small number of inspections) respect to the whole population, for the systematic neglecting of undamaged buildings.

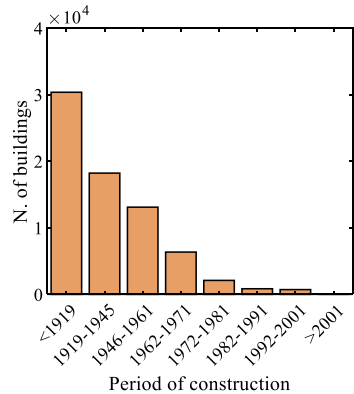
ISTAT data provide only information about number of storeys and construction. Further information about the vertical (good/regular or bad/irregular layout) and horizontal (i.e. vaults, flexible, semi-rigid or rigid slab) structural type are not obtainable. The percentage of these characteristics are retrieved from Da.D.O. data, taking advantage of their correlation with period of construction. In particular:

- for each period of construction, the subdivision between GQ and BQ masonry buildings is obtained from the percentage of Fig. 2c. Therefore, 71,662 BQ buildings are herein considered to represent the negative evidence of damage, which distribution is reported in Fig. 4.
- for each period of construction, the subdivision among horizontal types is obtained given the (BQ) masonry layout from the percentage of Fig. 3b.

Finally, the database considered in the following sections consists of:

- 18,869 *damaged* BQ masonry building deriving from Da.D.O. (*damaged database*);
- 71,662 *undamaged* BQ masonry building deriving from ISTAT 2001 census (*undamaged database*).

Fig. 4 Period of construction distribution for BQ undamaged masonry building deriving from ISTAT 2001 census



3 Data characterization of parameters collected after the 2009 L'Aquila earthquake

A large number of parameters plays a major role in the vulnerability analysis of masonry buildings, which characterization depends also by the available level of knowledge, among others: the design type, the period of construction, the number of storeys, the type of horizontal structure, the presence of tie rods and/or tie beams, the presence of structural interventions (Zuccaro 2004; Zuccaro and Cacace 2015).

The type of design is certainly one of the key parameters. Thus, a focus on technical laws enacted after main seismic events occurred on national territory, regulating the reconstruction process and planning of seismic risk mitigation interventions, is firstly done.

Then, an in-depth analysis of the final database will be performed considering the majority of parameters collected after the 2009 L'Aquila earthquake inspections, especially for what concerns the design typology and retrofit intervention.

At the beginning of twentieth century, two of the most powerful and devastating earthquakes ever occurred in Italy, struck several areas of southern and central Italy. The 1908 Messina caused about 80,000 victims in Sicilia Region and about 40,000 in Calabria Region, completely devastating several municipalities of southern Italy. Avezzano and Sora earthquake (13 January 1915) caused about 33,000 victims, strongly striking several municipalities in Abruzzi and Lazio Regions.

Prescriptions introduced by R.D.18/04/1909 n.193 (for territory affected by 1908 Messina earthquake) were also applied to Abruzzi municipalities through R.D. 29/04/1915 n.573: limitations about the total height and the inter-storey height, the restriction in the use of masonry vaults and of rubble stone masonry (except in case of one-storey buildings) and the prescription in the realization of a frame around masonry walls were compulsory for all seismically classified municipalities.

Therefore, the 1915 represents a milestone for building design in seismically classified municipalities, since the abovementioned prescriptions became compulsory, whereas no specific requirements existed before.

3.1 Code prescriptions related to seismic loads

R.D. 29/04/1915 n.573 prescribed also to consider the static forces equivalent to earthquake actions in building design, beyond those due to structural weights and live loads. The latter consisted in accelerations applied to structural masses of the building in both horizontal and vertical directions. Horizontal forces were equal to 1/8 of the storey weight for the first floor and 1/6 of the storey weight for the other floors of structures up to 10 m tall or 1/6 for taller buildings. The ratio between the base shear design, V_b , and the weight of the building, W , is variable between 0.125, for a single-storey building, to 0.152, for a 3-storey building, while for buildings with a number of stories greater than 3 it is equal to 0.166, assuming an interstorey height of 3 m.

In 1927 (R.D.L. 431/1927), the national territory was classified into two seismic categories: for the buildings located in seismic category I, horizontal forces to be applied in structural analysis were equal to 1/8 of the storey weight for structures up to 10 m tall or 1/6 for taller buildings. Instead, for the buildings located in seismic category II, horizontal forces were equal to 1/10 of the storey weight for structures up to 15 m tall or 1/8 for taller buildings. In seismic category II the ratio between the base shear design and the weight of the building is equal to 0.10 for buildings defined by a number of stories between 1 and 5, whereas for taller buildings is equal to 0.125, assuming an inter-storey height of 3 m.

Subsequently, the ratio between horizontal forces and storey weight was modified by several laws (R.D.L 640/1935; R.D.L. 2105/1937; L. 1684/1962). In fact, the ratio between the base shear design and the weight of the building became 0.10 and 0.07 in seismic category I and II, respectively, regardless of the number of stories of the building.

In 1975 (D.M. 3/3/1975 n.40) fundamental innovations were introduced in building design: the summation of lateral force distribution applied to each storey of buildings was defined as a function of the total weight of the structural masses, the fundamental period of the structure (through R coefficient), the soil compressibility (through ϵ coefficient), the possible presence of structural walls (through β coefficient) and, of course, the seismic action (through C coefficient). This law, through the definition of the R coefficient, introduced for the first time the concept of response spectrum. In fact, a constant response coefficient R was considered for fundamental periods not exceeding 0.8 s and a hyperbolic function thereafter. Moreover, a linear distribution of lateral seismic forces was introduced. The C seismic coefficient is equal to 0.10 and 0.07 for seismic category I and II, respectively.

After the 1980 Irpinia earthquake, a third seismic zone was introduced (D.M. 29/02/1984) and C seismic coefficients were assigned (D.M. 24/1/1986) to the three zones (0.10, 0.07 and 0.04 g respectively). In 1996, the limit state design method was introduced (D.M.16/01/1996) for the design in seismic zone.

On the other hand, seismic classification of the Italian territory remained unchanged until 2003. In fact, only after 2002 Molise earthquake, OPCM 3274/2003 classified the entire Italian territory as seismic, introducing also a fourth seismic zone. In particular, a different horizontal acceleration value a_g (i.e., that corresponding to 0 s period of vibration in the elastic spectrum) was assigned to each zone and the elastic response spectra were defined as a function of soil stratigraphy. Moreover, the limit states design method became mandatory for structural assessment.

Finally, a great innovation related to the definition of seismic loads was introduced by D.M. 14/01/2008. In fact, the seismic hazard parameters (i.e., input parameters of the response spectrum) were defined for a spaced grid of about 11,000 points on Italian territory taking advantage of the MPS04 model (Stucchi et al. 2004, 2011).

However, it should be noted that, despite the evolution of technical codes from 1915 until now regarding the definition of seismic forces as a function of site category and the evaluation of distribution of static forces, seismic design of a masonry building, explicitly taking into consideration seismic loads, became compulsory only after Irpinia 1980 earthquake (D.M. 9/01/1987). In fact, all prescriptions enacted until 1980 dealing with lateral loads in design/assessment referred to only moment-resisting frame structures. In other words, for over eight decades (1909–1986) the design of ordinary masonry buildings in seismic zone has been carried out in compliance with normative prescription about:

- maximum height or maximum number of storeys of the building according to seismic category of the construction site;
- minimum thickness of the walls depending on masonry type and seismic category;
- maximum distance between load-bearing masonry walls depending on seismic category;
- the band of use of some masonry textures.

These prescriptions together with further structural detailers are briefly provided in the next section.

3.2 Seismic prescriptions related ordinary masonry buildings

In Table 3 the evolution of seismic code prescriptions (1915–1975) related to ordinary masonry buildings is reported.

It has to be noted that only bricks and natural/artificial stone in squared blocks characterized by a rough surface were allowed in masonry constructions. Rubble stones were allowed only if spaced out by horizontal courses of bricks or by continuous bands of parallelepiped-shape rectangular stones or by R.C. tie beams. Starting from 1915 (R.D. 29/4/1915 n.573), it was forbidden in seismic zone the use of pebbles; this prescription was extended also to ordinary masonry buildings in non-seismically area after 1935 (R.D. 640/1935).

The maximum height (or the maximum number of storeys) for masonry buildings was settled depending on seismic category of the construction site. In 1915 (R.D. 29/4/1915 n.573), up-to-two-storey masonry buildings (i.e., characterized by a maximum height H_{max} of 7 m) were allowed in seismic area.

RD 13/03/1927 n.431 further modified this limitation as a function of seismic zones. In fact, construction up to two-storeys buildings (i.e., H_{max} of 8 m) was permitted in I seismic zone, and up to three-storeys buildings (i.e., H_{max} of 12.50 m) for II seismic zone. Subsequently (R.D. 22/11/1937 n.2105), three-storeys buildings (i.e., H_{max} of 12.50 m) were also allowed in I seismic zone and four-storeys buildings (i.e., H_{max} of 16.00 m) in II seismic zone, only in case of a regular texture (natural or artificial square bricks). Starting from 1962 (Law 1684/1962), the maximum number of storeys in the first seismic category was again limited to two storeys (i.e., H_{max} of 7.50 m) for I seismic zone and to three storeys (i.e., H_{max} of 11.00 m) for II seismic zone.

Then, the minimum thickness of masonry walls was settled depending on two factors: seismic category of the construction site and the quality of masonry texture. The prescribed value of thickness decreased with masonry quality and increased with the site's seismicity. For example, in the II seismic zone, a minimum thickness, s_{min} , of 30 cm (for top storey) was required in case of masonry walls made of bricks or natural/artificial squared blocks;

Table 3 Evolution of seismic prescriptions about ordinary masonry buildings

Scope of the law	RD 13/03/1927 n. 431		RD 25/03/1935, n. 640		R.D. 22/11/1937, n. 2105		L. 25/11/1962, n. 1684		D.M. 3/03/1975		
	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	
Seismic zone	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	
	NC zone		NC zone		NC zone		NC zone		NC zone		
Max building's height	7 m	8 m	12 m	8 m	12.50 m	16 m	7.50 m	11.0 m	16 m	11 m	7.5 m
Max number of storeys	2	2	3	2	3	4	2	3	-	-	-
Min. wall's thickness	40 cm	40-60 cm	30-45 cm	40-50 cm	40-50 cm	30-45 cm	40-50 cm	30-45 cm	30 cm	40 cm	40 cm
Max distance between walls	Every 5 m	Every 7 m	Every 7 m	Every 6 m	Every 6 m	Every 7 m	Every 6 m	Every 7 m	Every 7 m	Every 7 m	Every 7 m
Forbidden materials	Pebbles and irregular stone	Pebbles and irregular stone	Pebbles and irregular stone	Pebbles and irregular stone	Pebbles and irregular stone	Pebbles and irregular stone	Pebbles and irregular stone	Pebbles and irregular stone	Pebbles	Pebbles	-
Masonry vaults	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden except in case of tie rods	Forbidden except in case of tie rods	Forbidden
Cantilevered structures	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden
Pushing structures	Forbidden except in case of chain	Forbidden except in case of chain	Forbidden except in case of chain	Forbidden except in case of chain	Forbidden except in case of chain	Forbidden except in case of chain	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden
Tie beam											
							Width equal to wall's one; height min = 20 cm	Width equal to wall's one; height min = 20 cm	Width equal to wall's one; height min = 1/2 width	Width equal to wall's one; height min = 1/2 width	Width equal to wall's one; height min = 1/2 width

Table 3 (continued)

Scope of the law	R.D. 29/04/1915 n.573	RD 13/03/1927 n. 431		RD 25/03/1935, n. 640		R.D. 22/11/1937, n. 2105		L. 25/11/1962, n. 1684		D.M. 3/03/1975			
		1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT				
	Seismic zone	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	1° CAT	2° CAT	NC zone	s = 6	s = 9	s = 12
		-	-	Width equal to wall's one; height min = 20 cm		Reinforcement min = 4φ14 (long); φ5/30 cm (transv.)	Reinforcement min = 4φ12 (long); φ5/30 cm (transv.)	Reinforcement min = 4φ16 (long); φ6/25 cm (transv.)	Reinforcement min = 4φ16 (long); φ6/25 cm (transv.)	Number min of reinforcements = 4; ties every 25–30 cm; weight min of reinforcement = 50 kg/mc			
RC slab		-	-	Ribbed slab in two direction (thickness min = 5 cm)	-	Ribbed slab in two direction (thickness min = 5 cm)	-	Thickness min = 4 cm; reinforcement min = 1φ6/25 cm					Linked to tie beam

instead, in case of rubble stones masonry characterized by horizontal bricks courses, s_{min} was equal to 45 cm (R.D. 13/03/1927 n. 431). Moreover, an increment of 15 cm in thickness for each storey was required passing from the top to the ground storey. The latter prescription remained substantially unchanged until 1975.

A further prescription depending in seismic category dealt with the maximum distance between transverse load-bearing walls. Starting from 1927 (R.D. 13/03/1927 n.431), this distance in second seismic zone was limited to 7 m.

The above requirements were mainly related to geometrical prescriptions. Further prescriptions related to structural details were also enacted. For example, tie beam, characterized by a width equal to the whole depth of the wall and a height of at least 20 cm, became compulsory for all municipalities (seismically classified or not) with the enactment of R.D. 25/3/1935 n.640. Moreover, the latter also suggests for the first time for the use of RC slab in masonry buildings located in seismic zone in order to guarantee an effective connection between vertical and horizontal structures. It should be noted that the prescriptions about geometry of tie beam remained substantially unchanged in the following laws. Instead, minimum quantitative of reinforcements (introduced by R.D. 22/11/1937 n.2105) was modified several times, taking on different values as a function of the zone (seismically classified or not).

It is noteworthy that the use of masonry vaults was explicitly forbidden (except in case of vaults at the underground storeys equipped with tie rods) with R.D. 29/4/1915 n.573 and remained unchanged also for following codes until Law 1684/1962, which does not explicitly provide indication on this topic. Conversely, D.M. 3/3/1975 n.40 allows the use of masonry vaults, but only if adequately constrained by horizontal tie rods absorbing the horizontal thrust played by the former, voiding the overturning action otherwise acting on top of walls.

Then, the ban on the use of pushing structures was enacted for seismic zone for the first time with R.D. 18/4/1909 n.193, as a consequence of lesson-learned after earthquake occurrence (i.e., after the 1908 Messina earthquake). It is to be noted that this prescription became compulsory nationwide, then both for seismically classified and even not municipalities, with R.D. 25/3/1935 n.640.

All the above mentioned geometrical and structural requirements (wall thickness, presence of tie beams/tie rods, the removal of pushing structures, distance between walls, etc.) were explicitly recalled in Section 5 of the D.M. 3/3/1975 n.40, representing the main reference masonry building design until 1987. In fact, all subsequent codes (D.M. 19/6/1984; D.M. 24/01/1986) entirely confirmed these prescriptions.

Only with D.M. 9/1/1987, the design of the masonry building must take into consideration both vertical and horizontal actions, similarly to what prescribed since 1909 for the moment resisting frame structures.

3.3 Seismic prescriptions related to retrofit intervention on masonry buildings

The topics of existing buildings and of retrofit intervention for structural consolidation of masonry buildings damaged by earthquake was already handled by R.D. 18/4/1909 n.193. These interventions included the repair of damaged walls through the use of good quality mortar, the construction of external framed structure (columns connected with ties at each floor level), the removal of roof pushing structures, the insertion of circumferential or longitudinal ties and of keystones for vaulted structures and the replacement of stairs made of masonry. All these interventions were also adopted by subsequent codes, regulating the

consolidation of masonry buildings only in the seismic area (R.D. 29/4/1915 n.573; R.D. 13/03/1927 n.431) and then for the whole national territory (R.D. 25/3/1935 n.640; R.D. 22/11/1937 n.2105), regardless seismic classification.

All these prescriptions were explicitly recalled in Section 9 of the D.M. 3/3/1975 n.40, with slight modifications and/or integrations: the restoration of masonry walls could also be executed with concrete conglomerates or through the insertion of metal or reinforced concrete elements; the damaged slabs had to be replaced with steel or reinforced concrete slabs effectively encased and anchored within tie beams or floor beams; tie beams, if not present, had to be made at each storey.

After the 1980 Irpinia earthquake, an intense production of technical codes focused on the restoration/consolidation of buildings affected by earthquake occurred. In particular, D.M. 2/7/1981 n.593 regulated the post-earthquake reconstruction in the regions struck by the 1980 earthquake (Basilicata, Campania and Puglia), explicitly requiring seismic assessment of existing masonry buildings to be subjected to structural interventions.

In particular, the seismic assessment was required only if, in the as-built condition, the building did not meet all the requirements of Section 5 of the D.M. 3/3/1975 n.40 and the limitations relating to the height of the buildings (see Sect. 3.2). Moreover, particular emphasis was given to the connections between vertical walls and between the latter and the horizontal structures, to guarantee an adequate distribution of seismic forces. It was also requested that the horizontal structures have to ensure a rigid diaphragms behaviour. Several interventions were identified by the code to guarantee these performances: localized substitutions of damaged bricks (traditional “scuci-cuci”), grouting injections, insertion of reinforced concrete plates or steel grids, insertion of columns inside the walls, both horizontal and vertical tie rods, replacement of flexible slabs with steel or reinforced concrete slabs and removal of roof pushing structures. However, all these prescriptions, beyond seismic assessment if required, were mandatory only for municipalities affected by the Irpinia earthquake (as clarified by Law 30/7/1981 n.21745); vice versa, for the remaining municipalities seismically classified (therefore also for L’Aquila and surroundings municipalities) all the prescriptions reported in Section 9 of the D.M. 3/3/1975 n.40 were remained in force.

D.M. 24/01/1986 introduced for the first time a clear distinction between seismic upgrading and seismic improvement. The former, including a series of structural measures ensuring to the building to safely withstand design seismic actions, also required the execution of seismic assessment for the building. The latter, including one or more local interventions (the same defined in the DM 2/7/1981 n.593) aimed at improving seismic behaviour of the building, did not require the execution of seismic assessment for the building. Therefore, seismic assessment for masonry building is mandatory in all municipalities, regardless their seismic classification, starting from 1986, if seismic upgrading was required.

3.4 General features of post-earthquake damage data for L’Aquila and surrounding area

In this section, general features achievable from post-earthquake damage data for L’Aquila and surrounding area are analysed, among others the period of construction, the number of storeys, the quality of masonry vertical structures, the design type, the presence of structural interventions, the presence of tie rods and/or tie beams, the type of horizontal structure. A limited selection among all considered parameters for the vulnerability analysis

made further on should be inspired by a fair compromise ensuring the completeness of the analysis and not affecting the reliability of considered subsets of data. The characterization performed in this section aims at their selection.

The type of design is certainly one of the key parameters. To this aim, the subdivision between building constructed in seismic areas or not will be done by comparing the period of construction of each building with the year of first seismic classification of the municipality where it is sited. More precisely, being the available information on period of construction, given by AeDES form in term of fixed time interval period (i.e., < 1919; 1919–1945; 1946–1961; 1962–1971; 1972–1981; 1982–1991; 1992–2001; > 2001), its centroid is used for the comparison with the year of first seismic classification. To this aim, the software package ECS-it, *Evolution of the Italian Seismic Classification* (Del Gaudio et al. 2015) will be used, allowing the definition of the seismic classification of each municipality of Italian territory considering all (over 37) the classification codes enforced since 1909 to 2015. Clearly, if the period of construction precedes the year of the first seismic classification, the masonry building was designed according to seismic prescriptions (Seismic Designed—SD), vice versa the design class is GD (only Gravity load Designed), i.e. the masonry building was built without any normative prescription (before R.D. 640/1935) or was designed according to prescriptions compulsory nationwide (after 1935).

Figure 5 summarizes the evolution of the seismic classification laws (with different colors), together with CR values (with different line texture) for each Municipality of Abruzzi region. Note that the majority (33 out of 36) of the completely inspected municipalities have been seismically classified for the first time in 1915 (R.D. 29/04/1915 n.573), whereas *Campotosto*, *Calascio*, and *San Benedetto in Perillis* municipalities have been classified thereafter: the former in 1927 (R.D. 13/03/1927 n. 431) and the latter two in 1962 (Law 1684/1962). Masonry buildings therein located amount overall to 798 (434, 280 and 84, respectively) out of the total (18.869).

Thus, a clear relationship exists between period of construction and design type for the building stock investigated in this study, namely all masonry buildings dated back to before 1919 have been designed without any of the aforementioned seismic prescriptions (GD), conversely the majority of those constructed thereafter meet seismic requirements (SD).

Figure 6 precisely reports the strong relationship between period of construction and design types and resultant percentages for all of the 36 Municipalities under study. Obviously, the simple relationship that buildings constructed after 1919 can be considered seismically designed does not firmly apply to buildings sited in *Campotosto*, *Calascio*, and *San Benedetto in Perillis* municipalities, being seismically classified after 1915. In fact, a small sample (100) out of 798, although constructed after 1919 result gravity-loads designed, and will be discarded hereinafter for sake of simplicity. In quantitative terms, about 75% (14,256 buildings) of residential masonry buildings was constructed before 1919, without any seismic prescriptions, whereas the remaining 25% (4513 buildings) constructed thereafter follow seismic prescriptions thoroughly reported previously.

Moreover, the survey form allows gaining information also on the period when structural interventions are executed, beyond the original period of construction. Thus, the survey form for buildings subjected to structural interventions contains a double filled field regarding period, the oldest referring to its construction and the most recent referring to its retrofit. Obviously, this information is typically obtained by inspectors through a direct interview reliably granted by the owner. Additionally, the information on type of interventions, among injections or unreinforced coating, (H1), reinforced masonry or masonry with reinforced coating (H2), other or unidentified strengthening (H3), is also reported. Nonetheless, in almost all cases (about 98%) this information was not filled by surveyor.

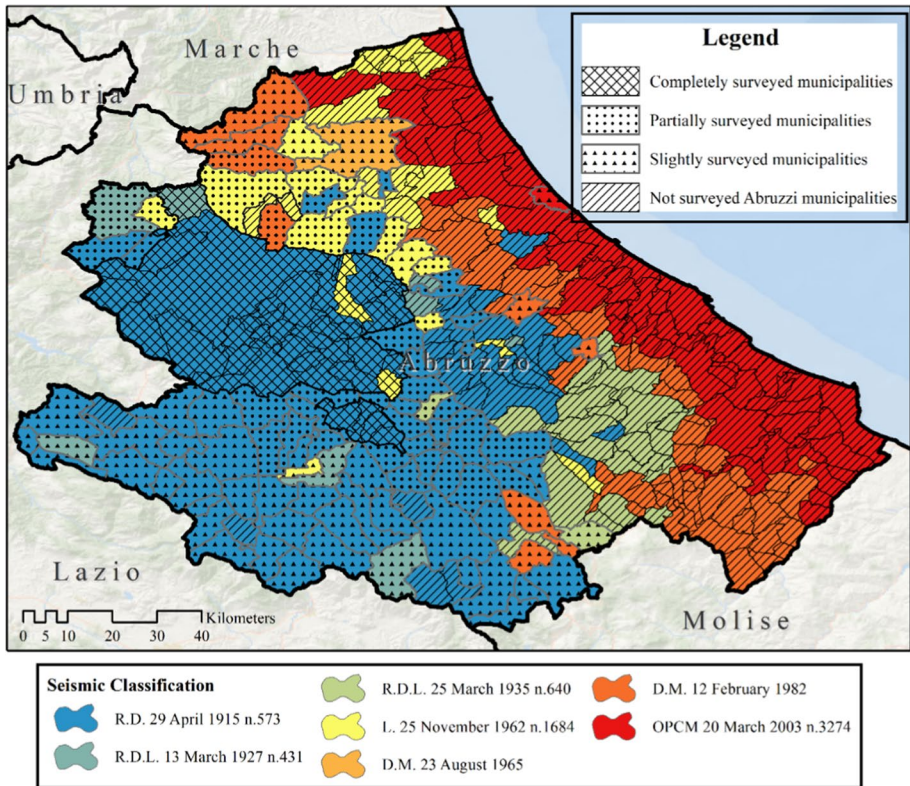


Fig. 5 First seismic classification for each municipality of Abruzzi region. Municipalities belong to damaged and undamaged database are characterized by different hatches

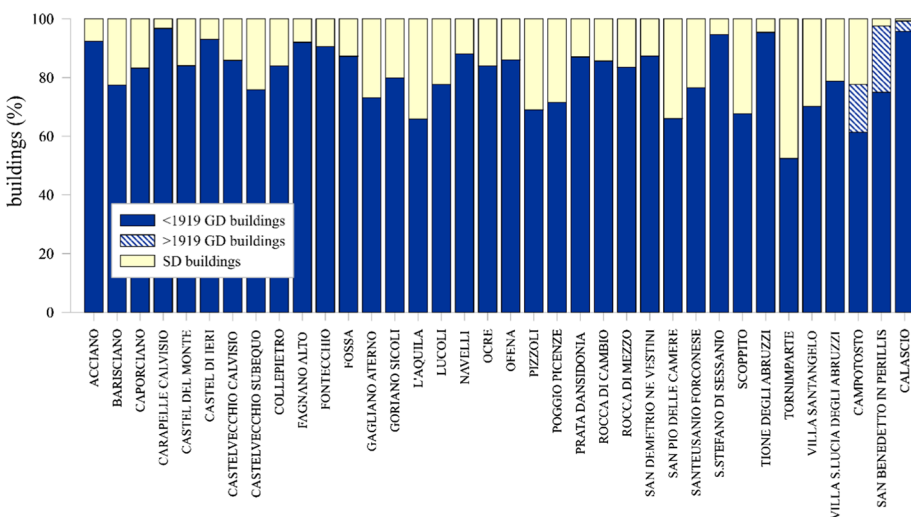


Fig. 6 Percentage of buildings belonging to the two considered design classes for all 36 completely inspected municipalities

Probably the rapidity required by emergency condition together with the way the inspections were conducted (only visual) did not allow the surveyors to precisely determine the kind of structural intervention, although they were aware that there had been. Thus, buildings, which survey form report double filled field regarding period, referring to construction and retrofit, are considered herein as been subjected to structural interventions over the years.

Finally, the original *damaged database* made up of 18,769 residential masonry buildings (with BQ vertical structure typology) shows that most of the buildings, about 94% (17,728) were built before 1961 (Fig. 7a). In particular, 76% of buildings (14,256) were constructed before 1919 (GD) and the remaining 24% (4513) thereafter (SD), as reported in Fig. 7a. In particular, about 88% of SD buildings dates back to the interval between 1919 and 1971.

Moreover, about 53% (9227) of buildings was subjected to structural interventions. Among those, 82% (7583) was constructed before 1919, amounting overall to 53% all buildings constructed in that period, whereas the vast majority of the remaining 18% was constructed between 1919 e il 1961, amounted to 40% of buildings constructed in that time, as reported in Fig. 7b, c.

The distribution of period of retrofit is reported with reference to buildings originally constructed before 1919 (Fig. 7d), between 1919 and 1945 (Fig. 7e) and between 1946 and 1961 (Fig. 7f). It is worth noting that for at least 80% of buildings the structural

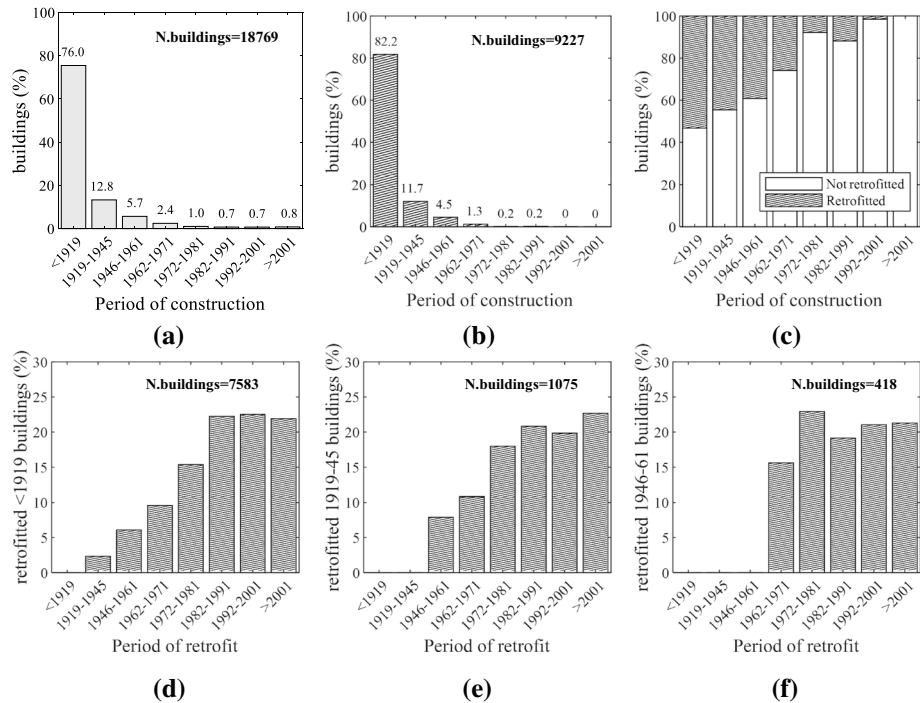


Fig. 7 Distribution of masonry buildings as a function of period of construction (a); distribution of retrofitted masonry buildings as a function of period of construction (b); percentage of not retrofitted and retrofitted buildings given the period of construction (c); distribution of retrofitted masonry buildings originally constructed < 1919 (d), between 1919 and 1945 (e), between 1946 and 1961 (f) for the damaged database

interventions were executed after 1971, regardless the original period of construction. In particular, the considerable efforts addressed to retrofit interventions after 1980 are probably due to the awareness that effective countermeasures were required against earthquakes, achieved after the lesson learnt of Irpinia 1980 and Abruzzi 1984 events. A clear attention on topic can be also highlighted in the context of legislative drafting, through the enacting of a specific regulation on strengthening, design, execution, and acceptance criteria of masonry buildings (D.M. 20/11/1987).

The influence of several other parameters (horizontal structural types, tie beams/tie rods, number of storeys), beyond retrofit interventions and period of construction, on vulnerability trend is treated hereinafter, possible leading to further declustering of data.

Firstly, the horizontal structural types are analysed: vaults (with or without ties), flexible-slab, semirigid-slab and rigid-slab, defined below V, F, SR and R, respectively.

Figure 8a shows the distribution of the horizontal structural types, given the period of construction, for not-retrofitted masonry buildings. Obviously, the different diffusion of the various horizontal types through the years produces sometimes singular assortments for some classes: for example, it is highly unlikely to show building dated back to before 1919 with rigid slab (recommended for the first time only with R.D.25/03/1935 n.640). Overall, the percentage of vaults and flexible slabs decreases increasing the period of construction, whereas, conversely, the percentage of semi-rigid and rigid slabs increases.

Figure 8b reports the distribution of the horizontal structural types, given the period of retrofit, for buildings subjected to structural interventions originally constructed before 1919. Generally speaking, a slight decrease (of approximately 10%) in the percentages of vaults and flexible slabs is observed compared to those observed for not retrofitted buildings constructed before 1919, compensated by an increase in semi-rigid and rigid slabs. Similarly, the distribution of the horizontal structural types, given the period of retrofit, for buildings constructed between 1919 and 1945 is reported in Fig. 8c. A further slight decrease (about 5%) in the percentages of vaults and flexible slabs respect to previous case (retrofitted buildings constructed before 1919), counterbalanced by the corresponding increase in percentage of rigid slabs.

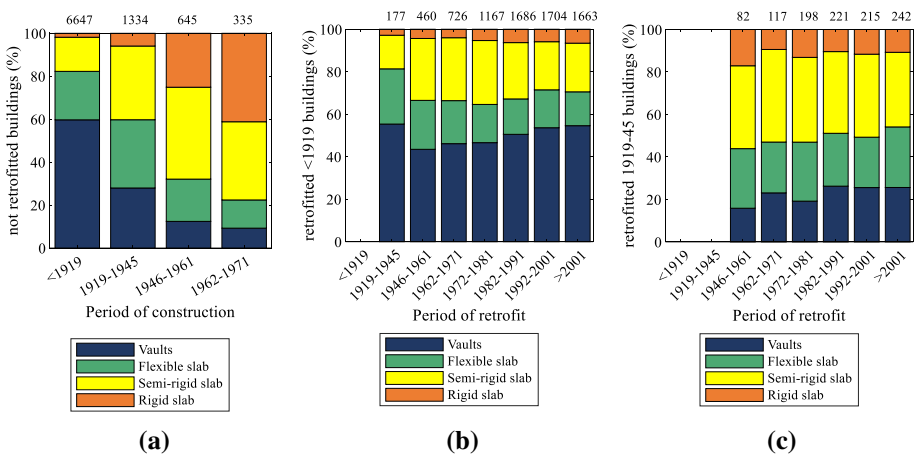


Fig. 8 Distribution of horizontal structural types given the period of construction for: not-retrofitted masonry buildings (a), retrofitted masonry buildings constructed before 1919 (b); retrofitted masonry buildings constructed between 1919 and 1945 (c)

Obviously, several other features could play a role in vulnerability assessment of masonry buildings, i.e. the presence of tie rods and/or tie beams and the number of storeys (Zuccaro 2004; Zuccaro and Cacace 2015).

The presence of tie beams/tie rods resulted not to have a significant influence on damage distribution of masonry buildings, particularly in masonry buildings with vaults (Del Gaudio et al. 2019) and/or for poor quality masonry (D’Ayala and Paganoni 2011; Sisti et al. 2018; D’Amato et al. 2020). Additionally, post-earthquake damage data shows that neither new buildings nor retrofitted buildings are characterized by high percentage of tie beams/tie rods. In fact, although the use of such a device became compulsory starting from 1935 (R.D. 640/1935) for all masonry buildings regardless the design type, percentages from 20% (for vaults or flexible slabs) up to 60–70% (for rigid slabs) of buildings equipped with tie beams/tie rods and constructed after 1946 can be observed in Fig. 9. This inconsistency with requirements enforced by technical codes can be justified by the fact that inspections were performed by means of rapid visual survey, which not always guarantee a complete awareness of all structural details, requiring sometimes more refined and invasive investigation (for example for the detection of tie beams), which are in poor agreement with the urgency required by the emergency phases.

On the other hand, the reduced increase in the percentage of buildings equipped with tie beams/tie rods between retrofitted and non-retrofitted buildings reveals that these devices represented a limited choice for structural interventions. In fact, the percentages of buildings with tie rods/beams increase from 17 to 23%, from 17 to 25%, from 19 to 30% and from 38 to 52%, respectively in case of vaults, flexible-, semi-rigid- and rigid slabs.

For all these reasons, the information on tie beams/rods is not considered in what follows.

Figure 10 provides the distributions of number of storeys, for each horizontal structural type, of retrofitted and non-retrofitted buildings, varying construction’s and retrofit’s

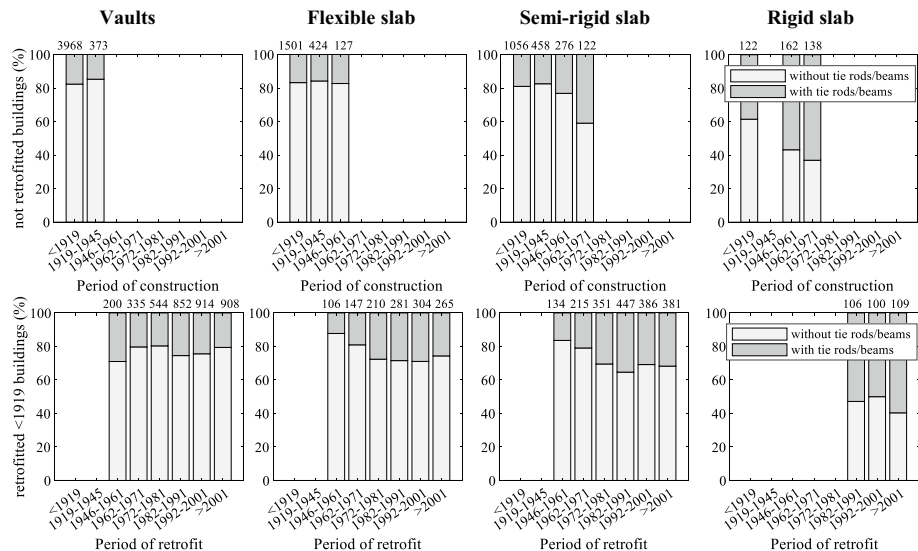


Fig. 9 Percentages of buildings equipped or not with tie rods/beams as a function of the horizontal structural type given the period of construction for all not retrofitted buildings (first row) and for all retrofitted buildings built before 1919 (second row)

Therefore, since no cross-correlation can be assumed to exist among considered parameters, any further partitioning of the sample as a function of the number of storeys is herein avoided.

Previously, the characterization of parameters available from survey inspections is fulfilled for *damaged buildings* collected in Da.D.O. database and suffering damage after the 2009 L'Aquila earthquake. A de-clustering analysis is then performed identifying homogeneous subset of buildings characterized by the different realization of such parameters.

The same analysis has also to be applied to the *undamaged* buildings, representing the negative evidence of damage. Nevertheless, the required level of knowledge is not available for this data, since their quantification arises from ISTAT 2001 census data allowing only to achieve information on structural (masonry and RC) types, period of construction and number of storeys. Thus, the available information on period of construction has been used as key to subdivide the original sample of 71,662 *undamaged* data as a function of the quality for vertical masonry structures, of the horizontal structural types and of the presence of structural interventions. In other words, it is assumed that the same percentage of abovementioned parameters for the *damaged* data given the period of construction can be extended also to *undamaged* data.

To this aim, the first subdivision between GQ and BQ masonry buildings was already done in Sect. 2.1 from the percentage of Fig. 2c, obtaining the original sample of 71,662 buildings. Then, the quantification of the *undamaged* subsets of buildings taking into consideration also the presence or not of retrofit interventions is done based on percentages reported in Fig. 7, and a further subdivision as a function of the horizontal structural types is made according to the percentages reported in Fig. 8. Moreover, in order to guarantee a strong consistency between *damaged* and *undamaged* data in term of seismic classification, the latter have been only restrained to those municipalities of Abruzzi Region which have been seismically classified for the first time in 1915. Thus, the *undamaged database* is composed by all buildings located in the 61 *slightly* or *not surveyed* municipalities classified in 1915, depicted in Fig. 12. In particular, only 22,270 out of 71,662 *undamaged* buildings coming from ISTAT 2001 census have been considered to characterize the negative evidence of damage, made up of about 54% (12,055) of buildings built before 1919 and the remaining 46% (10,212) of buildings built thereafter (Fig. 11a). Moreover, about 42% (9384) of buildings was subjected to structural interventions. Among those, 64% was constructed before 1919, amounting overall to 53% all buildings constructed in that period, according to distribution of *damaged* database shown in Fig. 7c. 23% and 10% were constructed respectively between 1919–1945 and 1946–1961, amounted to 45% and 40% of buildings constructed in that times, as reported in Fig. 7c. Clearly, the retrofitted buildings originally built before 1919, between 1919–1945 and between 1946–1961 are characterized by the same distributions of the period of retrofit shown in Fig. 7d–f for the *damaged* database.

4 Vulnerability curves

In this section, vulnerability curves are determined as a function of all the parameters achievable from post-earthquake damage data for L'Aquila and surrounding area, which, after the data characterization made in Sect. 3 given the available level of knowledge, allows to gain a complete vulnerability analysis not affecting the reliability of results due to an excessive subdivision of the original building stock. Among others, the period of

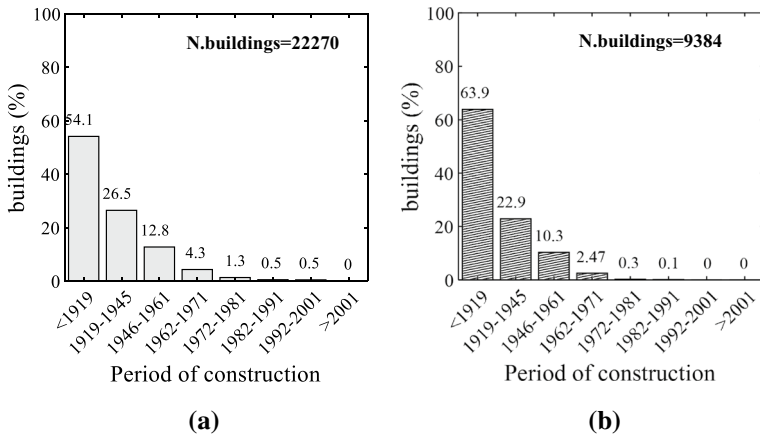


Fig. 11 Distribution of masonry buildings as a function of period of construction **(a)**; distribution of retrofitted masonry buildings as a function of period of construction **(b)** for the undamaged database

construction, the quality of masonry vertical structures, the design type, the presence of structural interventions, and the type of horizontal structure have been considered.

Least squared error method will be used to obtain lognormal distribution function parameters, best fitting with observed data.

Damage levels were defined according to EMS-98, considering 5+1 levels (from DS0—no damage—to DS5—collapse). To this aim, the (3+1) damage levels for vertical structures reported in the AeDES form:

- D0: no damage;
- D1: slight damage;
- D2–D3: medium-severe damage;
- D4–D5: very heavy damage

were converted in (5+1) EMS-98 damage states, using damage conversion rule (Table 4) proposed by Rota et al. (2008).

4.1 Intensity measure

Vulnerability curves in terms of mean damage and Peak Ground Acceleration (PGA), assumed herein as seismic intensity measure (IM), are obtained. Ground motion characterization is made using ShakeMaps in terms of PGA provided by Italian National Institute of Geophysics and Volcanology (Istituto Nazionale di Geofisica e Vulcanologia, INGV) (<http://shakemap.rm.ingv.it/shake/archive/>).

Figure 12 reports the ShakeMap of the April 6th, 2009 event, with the explicit location of each municipality belonging to *damaged* (i.e., completely surveyed municipalities) and *undamaged* (i.e., slightly and not surveyed municipalities, seismically classified in 1915) databases. This was generated through the software package ShakeMap developed by the U.S. Geological Survey Earthquake Hazards Program (Wald et al. 2006) specifically adapted for Italian events as thoroughly discussed in Michelini et al. (2008), through the

Table 4 Scheme of conversion of the damage levels for vertical structures into the EMS-98 Damage State

AeDES		EMS 98	
Damage level and extension	Damage description	Damage level	Damage description
D0	No damage	DS0	No damage
D1 (< 1/3)	Slight	DS1	Negligible to Slight damage
D1 (1/3–2/3)			
D1 (> 2/3)			
D2–D3 (< 1/3)	Medium-severe	DS2	Moderate damage
D2–D3 (1/3–2/3)			
D2–D3 (> 2/3)			
D4–D5 (< 1/3)	Very heavy	DS4	Very heavy damage
D4–D5 (1/3–2/3)			
D4–D5 (> 2/3)			
	Very heavy	DS5	Destruction

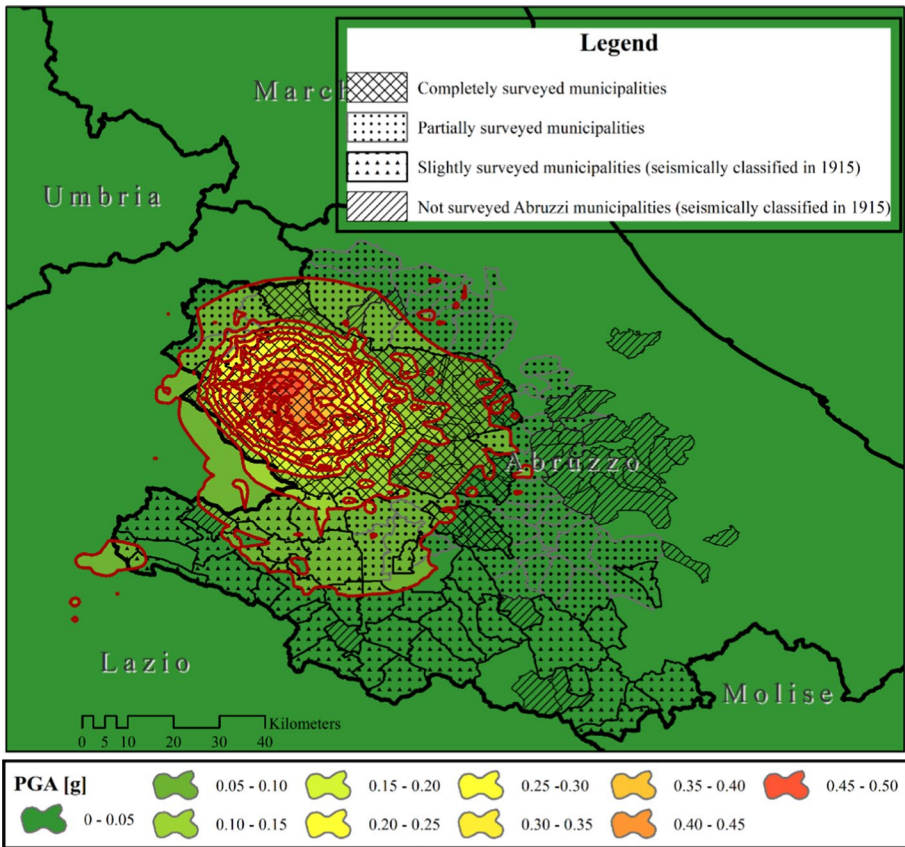


Fig. 12 ShakeMaps in terms of PGA (peak ground acceleration) of 2009 L'Aquila earthquake for all considered (completely, slightly and not surveyed) municipalities

use of the INGV broadband stations, besides strong motion data obtained from the Italian Strong Motion Network (*Rete Accelerometrica Nazionale, RAN*).

It should be noted that survey form used after the 2009 earthquake provide the coordinates (longitude and latitude) of each inspected building. Thus, a PGA value has been assigned to each building for *damaged database*, ranging from 0.03 to 0.50 g. Contrarily, being the building's location not available for *undamaged database*, deriving from ISTAT census, the value of PGA of the municipality's centroid was assigned to all buildings therein located, ranging from 0.01 to 0.08 g.

4.2 Vulnerability curves derivation for selected parameters

In this study, vulnerability curves will be determined starting from mean damage defined for each *j*th PGA bin as the weighted average of the damage distribution observed therein, $\mu_{D,j}$:

$$\mu_{D,j} = \frac{\sum_{i=0}^5 N_{Ed,j}(DS = ds_i)i}{N_{Ed,j}} \tag{1}$$

where $N_{Ed,j}(DS = ds_i)$ is the number of buildings within *j*th PGA bin and suffering damage level equal to ds_i ; $N_{Ed,j}$ is the total number of buildings within *j*th PGA bin.

Therefore, PGA bins of fixed width equal to 0.05 g, from 0 to 0.50 g, are used. So, damaged database covers all considered PGA bins, whereas undamaged database can be found only in the first two bins. The parameters defining the vulnerability curves are obtained using the LSE optimization technique (*Least Square Estimation*), minimizing the sum of the squares of the error between observed damage and lognormal cumulative function used to evaluate predicted mean damage values:

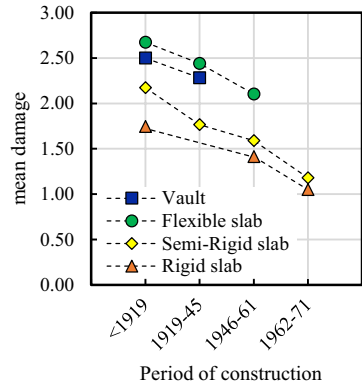
$$\operatorname{argmin}_{\mu,\beta} \left[\sum_j p(I_{PGA},\mu,\beta) - \frac{\mu_{D,j}}{5} \right]^2 N_{Ed,j} \tag{2}$$

In Eq. 2, the log-normal distribution function (CDF) is used as functional form p , which parameters are the logarithmic mean μ and standard deviation β . Furthermore, the potential inhomogeneity of database among the different bins is herein mitigated using the total number of buildings within *j*th PGA bin as weight in fitting procedure (see Eq. 1).

In the following, vulnerability trends will be investigated analysing (1) the mean damage, μ_D , calculated independent of the intensity measure, i.e. over the entire PGA range, and (2) the vulnerability curves, obtained as described above.

Figure 13 shows the μ_D trend for not-retrofitted masonry buildings as a function of period of construction, given the horizontal structural type. As above mentioned, the different diffusion of the horizontal structural types over the years could produce singular assortments in some sub-typologies. Thus, in Fig. 13 only subsets characterized by at least 100 buildings has been considered, assumed herein as the minimum acceptable sample size (Rossetto et al. 2013) for reliable vulnerability analysis. A decreasing trend of mean damage μ_D is observed, given the horizontal structural type, increasing the period of construction, probably due to the improvements in construction practices, to the enhancement of materials used in construction over the years, also related to the subsequent enactment of seismic prescriptions, or to the effect of degradation of building materials in existing structures. Strictly speaking, a straightforward comparison between gravitational and seismic

Fig. 13 Mean damage distributions for each horizontal structural type given the period of construction, for not-retrofitted masonry buildings



designed buildings, given the period of construction, cannot be done in this study, being all of them classified in the same years (i.e. in 1915). The effect of design type can only be analysed comparing μ_D values in the two adjacent classes of buildings constructed before 1919 and in the 1919–1945 time-interval. Due to proximity in period of construction it can be stated that the construction practice and building materials are nearly identical, whereas the difference in seismic vulnerability can be totally attributed to the effective contribution of the enactment of seismic prescription. In fact, as already discussed in Dolce and Goretti (2015), masonry buildings constructed in L'Aquila and adjacent municipalities after 1919 show a vulnerability comparable to that of moderate seismically designed typology of EMS98, thus confirming the effectiveness of seismic code prescriptions. Moreover, it can be noted that μ_D decreases increasing the slab stiffness (going from vaults to rigid slabs) given the period of construction, as already pointed out in previous studies (Del Gaudio et al. 2019; Rosti et al. 2020a, b).

Obviously, the trend reported in Fig. 13 does not explicitly consider the influence of intensity measure to which the building was subject. To this aim, vulnerability curves,

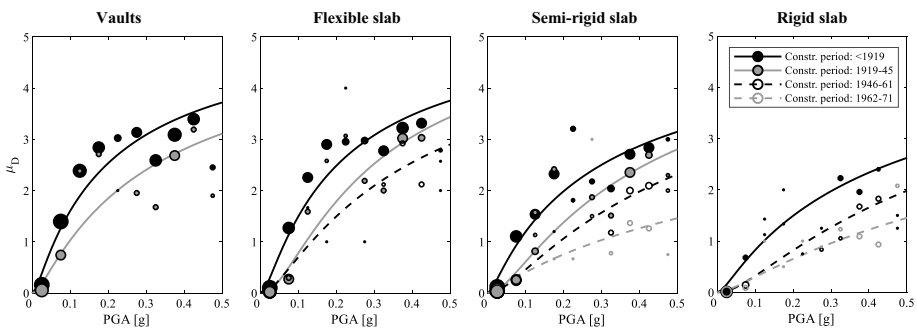


Fig. 14 Vulnerability curves for not-retrofitted masonry buildings varying the period of construction, for each horizontal structural type

considering the above defined minimum sample size of 100 masonry buildings to ensure their reliability, are introduced, see Fig. 14. The same hierarchy observed in Fig. 13, as concerns mean damage, is also confirmed in terms of vulnerability curves: in fact, a decreasing vulnerability is observed increasing the period of construction, given the horizontal structural type.

The comparison reported in Fig. 14 quantitatively shows the evolution in construction practices also affected by the introduction of succeeded code prescriptions in seismic areas (see Sect. 3.2).

Then, the influence of structural interventions for masonry buildings is analysed as follows, starting from buildings constructed before 1919, as shown in Fig. 15, which reports the mean damage, given the horizontal structural type, as a function of the period of retrofit. Furthermore, mean damage values for non-retrofitted masonry buildings constructed before 1919 are also reported in figure, assumed as reference for comparison. The reduction of mean damage due to the presence of structural interventions agrees with previous studies for the same area (Indirli et al. 2013; Rossetto et al. 2011; Azzaro et al. 2011; D’Ayala and Paganoni 2011; Zucconi et al. 2018; D’Amato et al. 2020).

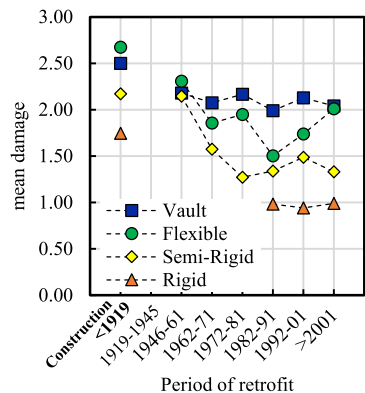
It can be noted an overall reduction of mean damage for retrofitted buildings, due to the structural interventions, compared to the values of those non-retrofitted and constructed in the same years (<1919), for each horizontal structural type. This reduction increases increasing the slab’s stiffness (i.e., going to vaults to rigid slabs). A constant decrease in μ_D of about 17% is observed in case of retrofitted masonry buildings with vaults, regardless the period of retrofit; similarly, a constant decrease in μ_D of about 40% is observed in case of buildings with rigid slabs. Lastly, in the case of flexible and semi-rigid slabs, a decreasing trend is observed until 1971, after which an irregular trend is observed; the overall reduction is, on average, 31% and 34%, respectively.

Moreover, also for retrofitted masonry buildings the same hierarchy, already noted for those as-built, in terms of mean damage with the horizontal structural type is observed, except for vaults and flexible slabs, where an reversed hierarchy occurs.

It has to worth noting that some irregular trends are expected in Fig. 15, since each subset could contain different kinds of interventions, which influence cannot be precisely determined due to the available level of knowledge achieved through the AeDES form.

Similarly, the influence of structural interventions on vulnerability curves of masonry buildings constructed before 1919 is shown in Fig. 15. Vulnerability curves are reported,

Fig. 15 Mean damage: influence of structural interventions for masonry buildings constructed before 1919, given the period of retrofit, for each horizontal structural type



for each horizontal structural type, as a function of the corresponding period of retrofit, namely in the four intervals 1946–1961, 1962–1971, 1972–1991 and > 1991. The selection of time intervals is justified by two aspects: (1) the great variability shown by the retrofitted buildings after 1971 with flexible and semi-rigid slabs in terms of mean damage (see Fig. 15), (2) the implications due to the enactment of D.M.1987 on the retrofit strategies of masonry buildings. Furthermore, vulnerability curves for non-retrofitted masonry buildings constructed before 1919 are also reported in figure, assumed as reference for comparison.

A clear effectiveness of structural interventions is shown, demonstrated by the lower vulnerability of retrofitted masonry buildings compared to those as-built, regardless the period of retrofit. In particular, with reference to the results of Fig. 16, it should be noted that:

- masonry buildings with vaults show a similar vulnerability behaviour for all the four periods of retrofit;
- masonry buildings with flexible slabs show a decreasing vulnerability with the period of retrofit until 1991. Surprisingly the vulnerability of buildings retrofitted thereafter results quite similar to those retrofitted during the period (1962–1971);
- the masonry buildings with semi-rigid slabs show a decreasing vulnerability with the period of retrofit until 1991. The vulnerability of buildings retrofitted thereafter results quite similar to those retrofitted during the period (1972–1991);
- the masonry buildings with rigid slabs retrofitted in the periods (1972–1991) and (> 1991) show a similar vulnerability.

The decreasing vulnerability of retrofitted masonry buildings with the period of retrofit is probably due to the improvement in construction practice and to an increasing attention to anchorage details, which make the difference between a more effective intervention or not (Indirli et al. 2013; Rossetto et al. 2011; Azzaro et al. 2011).

The influence of structural interventions on seismic behaviour of masonry buildings constructed between 1919–1945 and between 1946–1961, representing the most reliable subset as highlighted in Sect. 3.3, is herein evaluated given the horizontal structural type. Nonetheless, such an evaluation can not be done for all the periods of retrofit, since none of them overcome the assumed sample threshold of 100 buildings. Thus, both mean damage trends and vulnerability curves are presented considering a unique time intervals for retrofit.

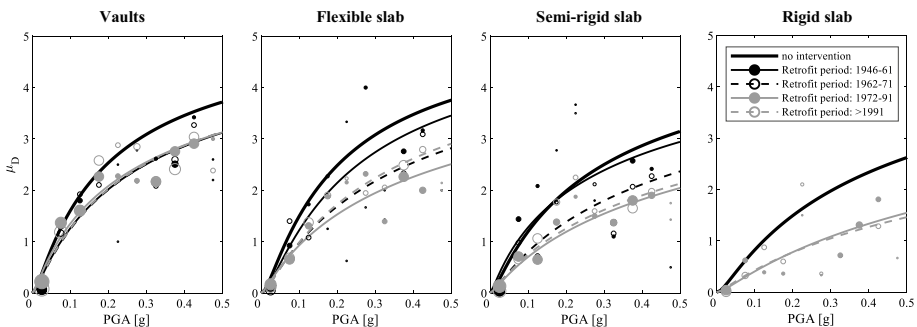


Fig. 16 Vulnerability curves: influence of structural interventions for masonry buildings constructed before 1919, given the period of retrofit, for each horizontal structural type

Fig. 17 Mean damage: influence of structural interventions for masonry buildings constructed (<1919), (1919–1945), (1946–1961), for each horizontal structural type

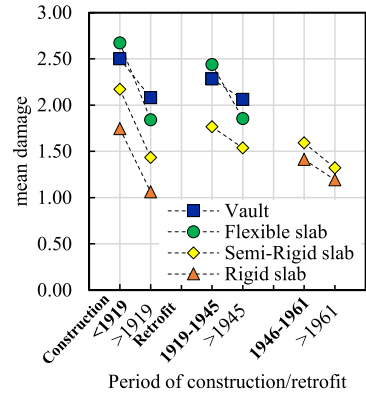


Table 5 Lognormal parameters of vulnerability curves for not-retrofitted masonry buildings as a function of structural typology and period of construction

Period of construction	Vaults		Flexible slab		Semi-rigid slab		Rigid slab	
	θ	β	θ	β	θ	β	θ	β
<1919	0.19	1.44	0.20	1.33	0.30	1.55	0.45	1.55
1919–1945	0.32	1.42	0.30	1.07	0.41	1.22	–	–
1946–1961	–	–	0.39	1.25	0.56	1.30	0.70	1.28
1962–1971	–	–	–	–	1.41	1.89	1.21	1.60

Figure 17 reports mean damage trends for non-retrofitted constructed <1919, between 1919 and 1945, between 1946 and 1961, besides those of constructed in the same years and retrofitted thereafter. The clear effectiveness of structural interventions is evident in any cases. An overall reduction of mean damage in case of retrofit ranging between 10 and 17% as a function of period of construction is observed for buildings with vaults and between 24 and 31% for flexible-slabs: Contrarily, for buildings with semi-rigid and rigid slabs a substantial reduction of the effectiveness of retrofit intervention can be observed.

The same comparison in terms of vulnerability curves for retrofitted and not buildings constructed between 1919 and 1945, between 1946 and 1961 is reported in Fig. 18, given the horizontal structural type. Vulnerability curves for retrofitted buildings is evaluated merging together damage outcomes for all retrofit’s periods.

As already shown, the effectiveness of retrofit intervention increases with slab’s stiffness, going from a negligible influence in case of vaults to a substantial influence in case of semi-rigid and rigid slabs. Probably, the effectiveness of structural interventions is limited in case of vaults or flexible slabs due to the absence of effective horizontal elements or devices acting to improve the box-like behaviour, without any specific intervention in this regard (introduction of stainless-steel tie rods, substitution of original flexible slabs).

Lastly, in Tables 5, 6 and 7 the lognormal parameters of obtained curves, i.e. the median θ and the logarithmic standard deviation β , are reported respectively for the

Table 6 Lognormal parameters of vulnerability curves for retrofitted masonry buildings built before 1919 as a function of horizontal structural typology and period of retrofit

Period of retrofit	Vaults		Flexible slab		Semi-rigid slab		Rigid slab	
	θ	β	θ	β	θ	β	θ	β
1919–1945	0.27	1.53	–	–	–	–	–	–
1946–1961	0.30	1.64	0.25	1.35	0.33	1.83	–	–
1962–1971	0.30	1.61	0.39	1.58	0.56	1.74	–	–
1972–1991	0.29	1.73	0.50	1.79	0.77	1.89	1.22	1.18
>1991	0.30	1.62	0.37	1.52	0.71	1.89	1.46	1.96

Table 7 Lognormal parameters of vulnerability curves for retrofitted masonry buildings built in 1919–1945 and 1946–1961 periods and then retrofitted as a function of horizontal structural typology

Period of construction	Vaults		Flexible slab		Semi-rigid slab		Rigid slab	
	θ	β	θ	β	θ	β	θ	β
1919–1945	0.34	1.36	0.45	1.30	0.59	1.42	–	–
1946–1961	–	–	–	–	0.86	1.60	1.37	1.92

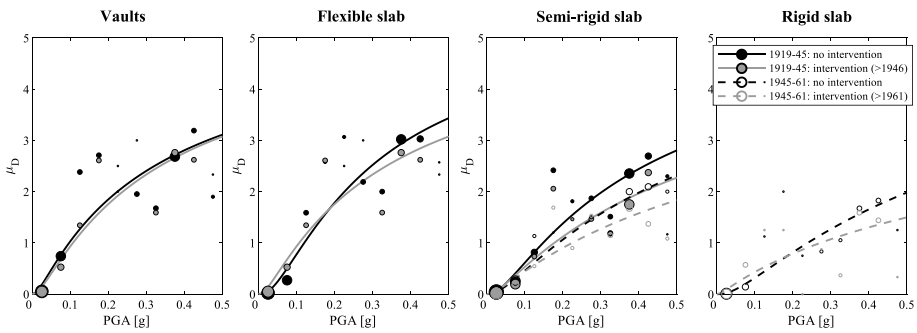


Fig. 18 Vulnerability curves: influence of structural interventions for masonry buildings constructed (1919–1945), (1946–1961), for each horizontal structural type

curves of Figs. 14, 16 and 18 as a function of structural typology and construction/retrofit period.

5 Conclusions

In this study, the evolution of seismic vulnerability on masonry buildings was evaluated through the derivation of empirical vulnerability curves, using damage data collected after 2009 L’Aquila earthquake and recently released by the Italian Department of Civil Protection.

A detailed data characterization was first done in terms of completeness level for given subsets of surveyed buildings, i.e. for each Municipality, and then, in terms of all the available parameters collected during the post-earthquake survey campaign. The output of this complex analysis was the so-called *damaged database*. Moreover, to prevent any possible bias due to the substantial lack of undamaged buildings during the inspections, a further *undamaged database* was considered, defined through the use of ISTAT 2001 census.

A selection of all the parameters achievable from post-earthquake inspections was made, aiming to perform a complete vulnerability analysis, nonetheless guaranteeing the reliability of results avoiding an excessive subdivision of the original building stock. Among others, the quality of masonry vertical structures, the period of construction and the design type, the presence of structural interventions, the type of horizontal structure, have been considered.

Therefore, vulnerability curves have been firstly derived as a function of period of construction, highlighting a clear evolution due to the improvements in construction practices and to the enhancement of materials used in construction over the years, also related to the subsequent enactment of seismic prescriptions, or to the effect of degradation of building materials in existing structures. Additionally, the substantial effect of the different design types is also addressed, by directly comparing vulnerability curves in the two adjacent time intervals before and after seismic classification, i.e. 1919, characterized by similar construction practice and building materials.

The main conclusions regarding the effectiveness of retrofit interventions that could be drawn from the comparison between mean damage values and vulnerability curves are as follows:

- A greater effectiveness is observed increasing the slab stiffness, probably because the box-like behaviour is ensured in case of rigid slabs, whereas, conversely, a partial or global overturning due to the absence of effective horizontal elements or devices have a greater probability of occurrence in case of flexible slabs.
- A decreasing vulnerability trend was shown as a function of period of retrofit, probably due to the improvement in construction practice and to the increasing attention to anchorage details.
- A reduced effectiveness of structural intervention for buildings designed for seismic loads compared to those designed for gravity loads only was shown, due to the fact that the former already met seismic prescriptions before they were retrofitted, thus limiting the contribution of structural intervention in reducing vulnerability.

The findings of this study suggest conducting further investigations on the evolution of vulnerability for gravity-loads designed buildings also, which in this study are limited only to those constructed before 1919. Nonetheless, the introduction of prescriptions also for ordinary buildings (in non-seismically classified area) together with the improvement in construction practices and the enhancement of building materials reasonably suggest similar trends of those observed in this study for seismically designed buildings.

Author contributions All authors contribute to the present research.

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Data availability The data analyzed in this work derives from the online platform Da.D.O. (*Database di Danno Osservato*, Database of Observed Damage) released from the Italian Department of Civil Protection (DPC). Further inquiries can be directed to the corresponding author or directly to the Italian DPC.

Declarations

Conflict of interest The authors declare that they have no conflict of interest.

Clinical trials registration Not applicable.

Plant reproducibility Not applicable.

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