

IMPROVED SEISMIC FRAGILITY AND VULNERABILITY CURVES FOR ITALIAN EXISTING MASONRY-INFILLED BUILDINGS

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Abstract: *In the last years, many research efforts have been focusing on the development of fragility and vulnerability functions of different building classes and structural typologies for large-scale seismic risk studies. For what concerns poorly-detailed masonry-infilled reinforced concrete (RC) buildings, although the role of masonry infills is well recognised, less attention has been paid to the quantification of the variability surrounding their mechanical properties and how this might affect the fragility curves and, consequently, the vulnerability curves for risk assessment. In such sense, this study deals with the development of fragility and vulnerability curves, for different damage states, of existing poorly-detailed masonry infilled RC frames. To do so, a large portfolio, representative of buildings designed according to the Italian codes in force between 1970 and 1980, is used as case-study. Building-to-building variability is also included through different geometrical configurations. Modelling uncertainty quantities, specifically derived for existing infilled RC frames, are used, including, in a thorough manner, the impact of the variability of the masonry infill typologies and the occurrence of RC frame shear failure. Multiple-stripe analyses are carried out and fragility curves are developed for buildings with different heights and geometrical configurations. Finally, vulnerability curves are derived using damage-to-loss models calibrated according to post-earthquake observation damage data from the 2009 L'Aquila earthquake and the actual repair costs monitored during the reconstruction process.*

INTRODUCTION

The seismic assessment of existing masonry-infilled reinforced concrete (RC) buildings is a highly relevant issue in Italy and other Mediterranean countries, particularly in regions where a large part of the built environment was not designed according to modern seismic codes. The results of experimental and numerical studies carried out in the last decade demonstrated the significant role played by masonry infills in the global and local response of RC buildings, which has also been observed during past earthquakes (Bosi et al., 2011). The interaction between the masonry infill panels and the surrounding frame, combined with the frequently poor seismic detailing in existing RC buildings, can lead to possible local shear failure of columns, short-column effects or soft-storey mechanisms during an earthquake (Braga et al., 2011; De Luca et al., 2018). Based on these considerations, the definition of material properties and numerical modelling approaches both of RC members and masonry infills is of paramount importance within the seismic assessment of infilled RC frames, both at single building and regional scales.

In the last decades, numerous efforts have been made for the development of fragility and vulnerability curves (Manfredi et al. 2023; Da Porto et al., 2021) to be used for seismic assessment at regional and/or national scales, to quantify the seismic risk of large building stocks, in terms of expected annual economic losses, downtime and fatalities. Moreover, regional and/or national-scale approaches are evermore used by governmental decision-makers who need to decide and justify the distribution of limited financial resources to reduce the overall seismic risk of the built assets, with a special focus on critical facilities. In this sense, a regional-scale approach should include and reflect, as much as possible, the geometrical and material ranges that are representative of the area of interest and accounting for the different sources of uncertainty. The HAZUS Earthquake Model (FEMA,2012), developed by the Federal Emergency Management Agency, is an example of a methodology developed in the past years to perform regional-scale studies. Such a model is designed to produce loss estimates for use by federal, state, regional

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and local governments in planning for earthquake risk mitigation, emergency preparedness, response and recovery.

Based on the above considerations, and given that neglecting the masonry infill variability might be reductive and lead to results that are not realistic, this study focuses on the development of updated fragility and vulnerability curves, accounting for the wide variability in the masonry infill properties and for more realistic epistemic uncertainty levels, for existing gravity load designed (GLD) infilled buildings. To do so, a large building stock (Mucedero *et al.*, 2021) is used, in which the uncertainty deriving from the variability on the properties and modelling assumptions of the masonry infills is accounted for. With respect to the available fragility and vulnerability curves proposed in the literature, those proposed herein account for the masonry infill variability and their influence on the seismic performance of existing GLD Italian buildings. Such an improvement in fragility and vulnerability curves leads to refined seismic risk estimates of both single buildings and building portfolios.

RESEARCH METHODOLOGY

Uncertainty characterisation

Despite the high-level of uncertainty surrounding the masonry infill properties, constant mechanical and geometrical properties are typically assumed in seismic risk assessment studies. Furthermore, a proper identification and propagation of uncertainty in the collapse assessment of existing structures are extremely important when detailed nonlinear structural analysis methodologies are adopted; hence an explicit consideration of the variability in the masonry infill characteristics should be considered for a refined collapse and loss estimation. Recently, to overcome the lack of in-situ test results on masonry infills, a macro-distinction approach of different masonry infill typologies was proposed by Mucedero *et al.* (2020), based on masonry infill strength. Five masonry infill typologies were selected as representative of the existing masonry infill typologies used in RC residential buildings, and their representativeness was proved with respect to the ranges of masonry infill properties provided in an experimental tests database available in the literature (De Risi *et al.*, 2018; Furtado *et al.*, 2020).

The effects of the high-level of uncertainty surrounding the masonry infill properties on the quantification of the epistemic uncertainty was investigated in few research studies. Although several studies investigated the impact of different sources of epistemic and aleatory uncertainty on the seismic assessment of RC buildings, little attention was paid to the impact of the uncertainty related to the variability of masonry infill properties, in a thorough manner. Recently, O'Reilly and Sullivan (2018), using the Correlated Latin Hypercube Sampling method, proposed by Olsson *et al.* (2003), have investigated and quantified the uncertainty associated with different modelling parameters for existing RC frames in Italy, with and without masonry infills. More recently, Mucedero *et al.* (2022) further integrated the estimation of modelling uncertainty in existing buildings, using a case-study masonry-infilled RC frame from an extensive building stock (Mucedero *et al.*, 2021), representative of existing RC frames built in Italy between 1970 and 1980, and covering some important aspects that were unaddressed by previous research studies. The main steps forward made by Mucedero *et al.* (2022) were related to: (i) a more refined modelling technique (three struts modelling approach) of the masonry infills; (ii) a wider spectrum of characteristics of the infill typologies that can be found in existing buildings; and (iii) the possible shear failure of RC members. More details are provided in Mucedero *et al.* (2022).

Adopted approach

With such considerations in mind, this study proposes improved fragility and vulnerability curves for existing infilled buildings in Italy, accounting for the masonry infill-related uncertainty in a more thorough manner, as described above. The points briefly discussed below summarise the adopted research methodology:

- The building portfolio developed in Mucedero *et al.* (2021) is used as case-study, which is considered representative of RC GLD buildings built in Italy between 1970 and 1980;
- Characterisation of the seismic hazard for the city of L'Aquila (Italy) and selection of suitable hazard-consistent ground motion sets. The record sets are selected using an average spectral acceleration (AvgSa)-based procedure (Khorangi *et al.*, 2017);
- Nonlinear time history (NLTH) analyses are carried out on the building portfolio, following a Multiple Stripe Analysis (MSA) approach, to estimate its seismic performance, for ten selected intensity measure (IM) levels;

- Derivation of fragility parameters through the MSA results. The median AvgSa and the record-to-record variability (β_{RTR}) values, at each damage state, were modified to account for the epistemic and the damage threshold uncertainties. The refined dispersion set for the epistemic uncertainty proposed in Mucedero *et al.* (2022) is employed herein, whereas the damage threshold uncertainty is defined according to Manfredi *et al.* (2023);
- Building-based vulnerability assessment, which is the most used approach for generating vulnerability curves, is used herein; damage-to-loss models calibrated according to post-earthquake observation damage data from the 2009 L'Aquila earthquake were used.

CASE-STUDY BUILDINGS

The selected case-study masonry-infilled RC frames were taken from buildings with two to six storeys. The design was carried out using a simulated design approach to reproduce existing GLD buildings in Italy prior to the introduction of modern seismic design, thus according to the codes in force in Italy between 1970 and 1980 (D.M. 30/05/1972). The mechanical properties of the materials used in the selected construction period were obtained from the results of in-situ tests and from data collected in laboratory archives (Verderame *et al.*, 2011; Masi *et al.*, 2014). For what concerns the mechanical characteristics of the steel reinforcement, detailed information is provided in Verderame *et al.* (2011), in which the steel rebars' properties used in Italy between 1950 and 1980 were statistically collected from archive data of laboratory steel testing. Considering the wide range of steel properties adopted in the reference construction period, at least five, it was decided to use an average value of 350MPa for the design yield strength, whereas the ratio between ultimate and yield strength was assumed as 1.4 (Verderame *et al.*, 2011). The diameter of the longitudinal bars was taken as 14mm, according to the mean value provided in Verderame *et al.* (2011). For what concerns the concrete strength, exhaustive information is presented in Masi *et al.* (2014), from a large database of test results on concrete cores extracted from about 300 RC public buildings located in the Basilicata region (Italy), for which the mean strength and dispersion for different construction periods were provided. The test results showed a mean value of concrete strength in the construction period 1972–1981 of 21.03MPa, thus, for simplicity, 20MPa was considered as the mean concrete strength in the period 1970-1980. The total storey loading was taken as 9kN/m² and 7kN/m² for intermediate and roof stories, respectively, in addition to the own weight of the structural elements. More details on the design process and the full building portfolio can be found in Mucedero *et al.* (2021). A summary of the geometrical and material properties of the case-study frames, as well as the total storey loads, is provided in Table.1.

Description	Variable	Value or Range
Building length	L_x	10 m
Span length	b_x	[3-4] m
Number of stories	n_s	[2-6]
Interstorey height	h	3.0 m
Steel yield strength	f_y	350 MPa
Longitudinal rebar diameter	ϕ_l	14 mm
Stirrup diameter	ϕ_s	8 mm
Concrete compressive strength	f_c	20 MPa
Concrete cover	c	20 mm
Column height*	h_c	[300-550] mm
Beam height	h_b	[300-400] mm
Column or beam width	b_c or b_b	300 mm
Typical story load	q_t	9kN/m ²
Roof story load	q_s	7kN/m ²

*Higher cross sections are referred to columns at lower stories of high-rise buildings (five- and six-story buildings)

Table.1 Geometrical and material properties of the case-study frames, together with the total story loads List of adopted random variables (RVs).

Seismic hazard and ground motion record selection

OpenQuake (Silva *et al.*, 2014) was used to perform the seismic hazard computations, adopting the SHARE project (Giardini *et al.*, 2014) source model and the ground motion model proposed by Boore and Atkinson (2008). For the purposes of ground motion selection, the case-study

building frames were assumed to be located in L'Aquila (Italy), an Italian city with a very high seismic hazard level and for which past earthquake surveys are available, thus rendering the results obtained herein particularly useful for future validation studies. The corresponding hazard curve for AvgSa (0.01÷2.0s), considering a soil type C, was derived and the main parameters are reported in Mucedero *et al.* (2022). Ten intensity measure (IM) levels were adopted and a total of 30 non-frequent ground motion records were selected for each IM level from the PEER NGA-West database (Ancheta *et al.*, 2013). More details on the record selection approach, which employs the conditional spectrum (CS) and the spectral acceleration (geometrically) averaged over a period range as IM, are provided in Kohrangi *et al.* (2017). The adopted period range was based on a preliminary assessment of the dynamic properties of the building portfolio. Further details on the dynamic properties of the building portfolio are reported in Mucedero *et al.* (2021).

Numerical model

An accurate numerical model for old existing buildings should be able to consider many structural response aspects, such as material and geometrical nonlinearity, rebar slippage, flexible beam-column joints, damage of non-ductile and poorly detailed RC frame members, shear failure due to deficiencies in stirrup spacing or lack of concrete core confinement. In order to account for these aspects, the numerical model approach proposed by Mucedero *et al.* (2020), whose accuracy was proved using experimental testing results, was adopted. The case-study buildings were modelled following a 2D approach, considering the regularity, in plan and elevation, of the adopted buildings, for which torsional effects are not expected. Under such regularity conditions, the 3D and 2D modelling approaches would thus provide similar results, in terms of dynamic response, hence the 2D approach was preferred to reduce the computational onus.

FRAGILITY AND VULNERABILITY CURVES

Nonlinear time history (NLTH) analyses were carried out on the building portfolio, following the MSA approach, to investigate its seismic performance, for the ten selected IM levels. Based on the geometrical and masonry infill variability accounted for in the building portfolio, a total of 540 RC infilled frames were analysed, leading to a total of 162'000 NLTH analyses. More details on the structural response, in terms of interstorey drift ratio (IDR) and peak floor acceleration (PFA), can be found in Mucedero *et al.* (2022).

Damage states

In the development of fragility curves, a proper relationship between the Engineering Demand Parameter (EDP) and the corresponding building performance is needed. The definition of EDPs generally refers to member forces/deformations, displacements (e.g., IDR) or energy-based indicators (e.g., Park and Ang 1985). In the case of RC members with low ductility and infilled frames (namely with drift-sensitive components), the IDR is very often chosen as EDP.

In the case of masonry infilled frames, the first three damage levels are generally related to damage of the masonry infills, whereas for the most severe damage levels (i.e., DS4–DS5) only structural element performance is generally considered. In the literature, there are numerous studies addressing the definition of damage control for masonry infills in the design and assessment of RC frame structures, in which threshold values for damage identification based on experimental tests are proposed. One example is the large database of experimental data that was used by De Risi *et al.* (2018) to develop fragility curves in terms of IDR for the damage levels consistent with the EMS-98 definition. This database collected 75 experimental tests on specific brick typologies, such as hollow clay bricks. Similarly, Cardone and Perrone (2015) developed fragility curves and loss functions for masonry infill walls by using the results from 19 different experimental investigations. Hak *et al.* (2012) proposed three limit states for medium and strong masonry infills, which were defined through numerical analysis and calibration of an equivalent diagonal strut model with available test results. In the work of Morandi *et al.* (2018), the in-plane cyclic tests on RC frames with strong masonry infilling were presented, together with the definition of three limit states based on the increasing extent of infill damage due to in-plane actions. Table.2 summarises the most recent IDR thresholds for each limit state available in the literature.

Regarding the most severe damage levels (i.e., DS4–DS5), different proposed threshold values can be found. In FEMA 356 (1997), an IDR value equal or higher than 4% is considered as building collapse, whereas in Vision 2000 (SEAOC, 1995) the value is 2.5%. Ghobarah (2004) provided different IDR values for the collapse as a function of the structural typology: (i) $IDR \geq 3\%$ for bare frames; (ii) $IDR \geq 0.8\%$ for infilled frames; (iii) $IDR \geq 3\%$ at the ground floor and $IDR \geq 0.8\%$ at the remaining floors for pilotis frames (thus in between bare and infilled frame values).

According to Masi *et al.* (2015), DS4 is achieved for $1.2 \leq IDR \leq 3.5\%$, as a function of the design level and the construction period. Table.3 presents the IDR thresholds assumed in this study. Four damage stages are identified, as has also been done for the European vulnerability database developed as part of the European Seismic Risk Model (ESRM20). The IDR threshold values for DS1, DS2 and DS3 are in line with the values proposed by Hak *et al.* (2012), Cardone and Perrone (2015) and Morandi *et al.* (2018), due to similarity with the selected masonry infill typologies, and slightly higher than those proposed by De Risi *et al.* (2018). The IDR threshold value (2.0%) for DS4 is close to the mean IDR value (2.35%) of the threshold range outlined in Masi *et al.* (2015).

Reference	Statistical parameters	Damage state		
		DS1	DS2	DS3
De Risi <i>et al.</i> (2018)	Median [%]	0.08	0.33	0.83
	Log. standard deviation	0.71	0.41	0.48
	16 th [%]	0.04	0.22	0.51
	84 th [%]	0.16	0.50	1.34
Cardone and Perrone (2015)	Median [%]	0.19	0.57	1.10
	Log. standard deviation	0.37	0.40	0.32
	16 th [%]	0.13	0.38	0.79
	84 th [%]	0.28	0.85	1.51
Masi <i>et al.</i> (2016)	Median [%]	0.20	0.64	1.31
	Log. standard deviation	0.09	0.32	0.43
	16 th [%]	0.11	0.32	0.89
	84 th [%]	0.28	0.96	1.73
Hak <i>et al.</i> (2012)	Median [%]	0.20	0.30	1.00
Morandi <i>et al.</i> (2018)	Median [%]	0.30	0.50	1.75

Table.2 Summary of the most recent IDR thresholds for the identification of masonry infill damage states.

Damage State	DS0 (None)	DS1 (Slight)	DS2 (Moderate)	DS3 (Extensive)	DS4 (Collapse)
IDR (%)	0.1	0.2	0.5	1.0	2

Table.3 IDR values related to damage levels assumed in this work.

Fragility curves

For each NLTH analysis carried out on the building portfolio frames, the maximum IDR value along the height is evaluated and, using such values, Equation 1 is applied for each damage state (Table.3) to derive fragility curves (e.g., FEMA 2018):

$$P(D_s|IM) = \Phi \left[\frac{1}{\beta_{RTR, D_s}} \cdot \ln \left(\frac{IM}{IM_{D_s}} \right) \right] \quad s = 1, \dots, 4 \quad (1)$$

where $P(D_s|IM)$ is the probability of exceedance of the s -th damage level D_s given an IM level, Φ is the standard normal (Gaussian) cumulative distribution function, and β_{RTR, D_s} and IM_{D_s} denote the logarithmic standard deviation and median values, respectively. In Equation 1, the logarithmic standard deviation term (β_{RTR}) accounts only for the record-to-record variability, which is obtained from the NLTH analyses. For a comprehensive consideration of the uncertainty, the epistemic uncertainty (β_{MDL}) and damage level definition uncertainty (β_{DS}) should be added. In this framework, each factor contributing to the overall uncertainty (β_{TOT}) is considered as independent, thus it is computed through a square root sum-of-the-squares (SRSS) rule, as per Equation 4.10:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{MDL}^2 + \beta_{DS}^2} \quad (2)$$

where β_{RTR} accounts for the record-to-record variability, β_{MDL} is the epistemic uncertainty, β_{DS} is the uncertainty in the definition of damage levels. The epistemic uncertainty (β_{MDL}) is set equal to the values proposed in Mucedero *et al.* (2022). Regarding the damage threshold uncertainty (β_{DS}), which accounts for the definition of the limit levels, it is set equal to 0.4, according to Manfredi *et al.* (2023). Finally, in order to eliminate intersection of fragility curves related to two or more damage states, the total dispersion value (β_{TOT}), as shown in Table.4, is adopted for all the

damage states, using the equations outlined in Porter et al. (2007). The resulting fragility curves, for each damage state, are presented in Figure.1, whereas the mean values, for each damage level, and the β_{TOT} are summarised in Table.4.

Buildings	ϑ (DS0)	ϑ (DS1)	ϑ (DS2)	ϑ (DS3)	ϑ (DS4)	β_{TOT}
Two storeys	1.25	1.55	1.69	1.77	1.83	0.90
Three storeys	0.38	0.89	1.06	1.22	1.46	1.17
Four storeys	0.28	0.51	0.65	0.76	1.05	1.02
Five storeys	0.21	0.39	0.55	0.65	0.82	0.90
Six storeys	0.15	0.32	0.44	0.55	0.78	0.88

Table.4 Fragility parameters for each damage state, as a function of the number of storeys.

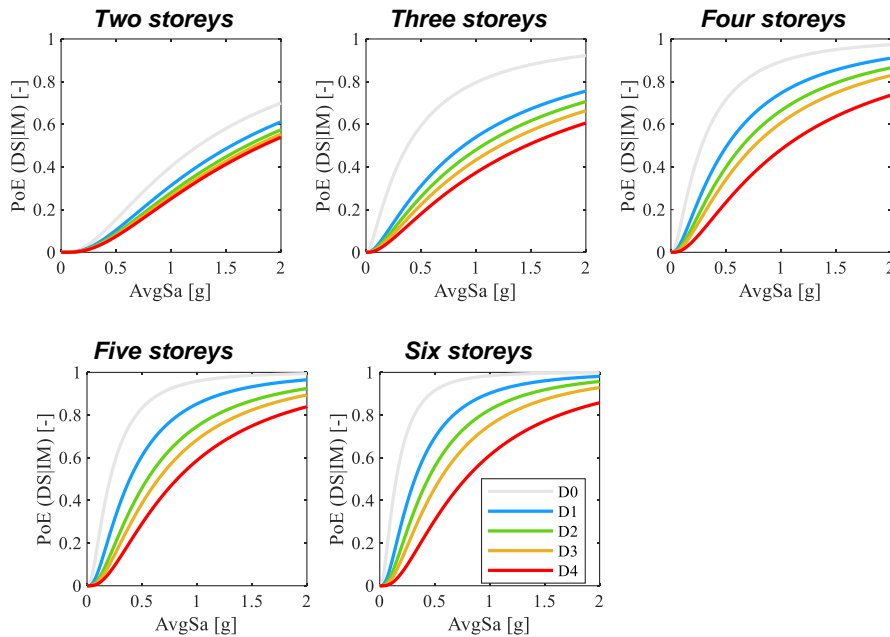


Figure.1 Fragility curves for each damage level, as a function of the number of storeys.

Given the structural deficiencies of the selected building typologies, such as buildings with poorly detailed structural members and with possible premature shear failure of RC columns, an additional consideration related to the relationship between the EDP and the corresponding building performance is needed. In particular, as for most of the available studies (e.g., Manfredi et al., 2023; Masi, 2015), IDR is chosen as EDP. It is worth mentioning that, in case of poorly-detailed RC infilled frames, with medium-strong masonry infills, premature shear failure of columns can occur and it can strongly affect the seismic performance of the buildings (Mucedero et al., 2022; Blasi et al., 2018). Moreover, shear failure could take place for very low IDR values, depending on the stiffness/strength of the masonry infills and the shear capacity of the RC columns. This means that the IDR value at which premature shear failure may happen is not necessarily close to the occurrence of the D4-D5 limit states. In this sense, the fragility curves using the EDP for first shear failure as collapse criterion are investigated and presented in Figure.2. Such criterion to identify the collapse is more in line with the recommendations of NTC-18, according to which the collapse limit state is achieved when the first structural element in a structure reaches failure through either a brittle or a ductile mechanism, which does not necessarily represent a global failure of the building. The results (Figure.2) clearly show how, considering the first shear failure as the achievement of the collapse limit state (in line with the recommendations of NTC-18), the corresponding probability of exceedance is generally higher than that obtained for DS1, hence before the slight damage of masonry infills and partition walls occurs.

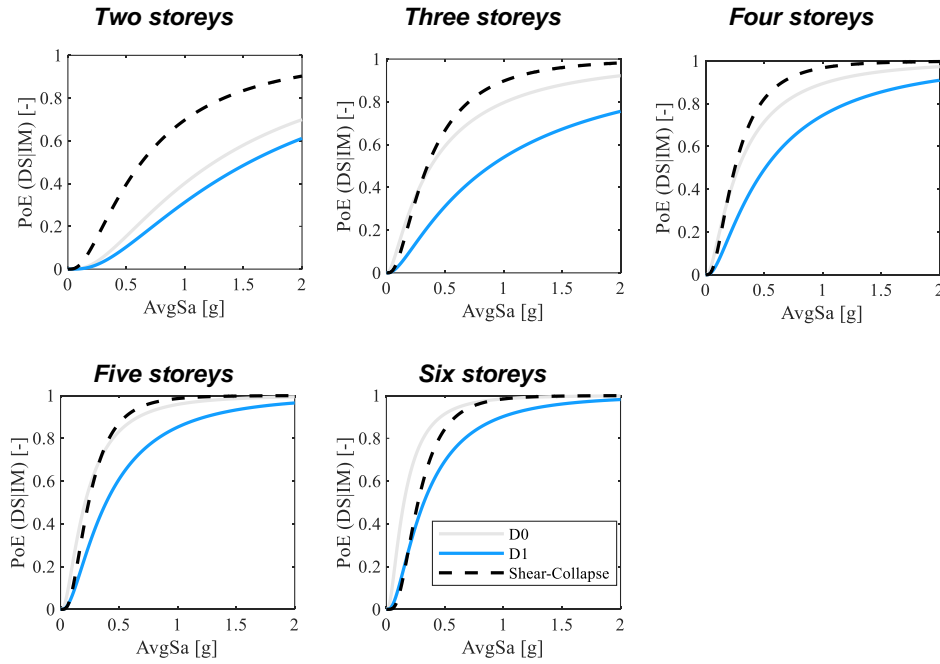


Figure.2 Comparison of fragility curves using two different EDPs.

Vulnerability curves

Two approaches can be used to generate vulnerability curves, depending on the available data and the required degree of refinement, briefly described as follows:

- **Building-based vulnerability assessment:** it is the most used approach for generating vulnerability curves. The vulnerability functions are derived using damage-to-loss functions and fragility curves; in specific, vulnerability functions are obtained by convolving building fragility curves (derived at each damage level) with the cumulative repair cost associated with a given damage state ($D_{s,i}$). The simplicity in implementation of the method, the low calculation effort and the availability of detailed data encourages its use in case of loss assessment studies at regional/national level;
- **Component-based vulnerability assessment:** it is more complex and requires more data with respect to the previous one. The approach is extensively described in ATC-58 (FEMA P-58, 2012). The vulnerability functions are obtained by correlating the component level-based drifts/accelerations directly to loss and it is generally used at single building level, obtaining vulnerability curves for both the storey- and building levels. The analyst has to collect the relevant information on components, thus creating a building inventory, and to perform detailed analysis, which are generally time consuming and expensive.

Considering the use of the developed functions for large-scale purposes, the building-based vulnerability assessment approach, which is further described next, was deemed the most suitable one. The total probability theorem can be used for the transformation of the fragility into vulnerability curves. In this sense, Equation 3 can be used:

$$E(C > c|IM) = \sum_i^n E(C > c|d_{s,i}) \cdot P(d_{s,i}|IM) \quad (3)$$

where n is the number of damage states considered, $P(d_{s,i}|IM)$ is the probability of a building sustaining damage state $d_{s,i}$ given an IM level; $E(C > c|d_{s,i})$ is the complementary cumulative distribution of the cost given $d_{s,i}$, and $E(C > c|IM)$ is the complementary cumulative distribution of cost given $d_{s,i}$ an IM level. The vulnerability curves are then defined through the calculation of the damage probabilities obtained from the fragility curves for specific levels of intensity. Hence, the mean, $E(C > c|IM)$, and the variance, $\text{var}(C|IM)$, of the vulnerability function are obtained by using Equations 4 and 5, which are employed n -times for increasing values of IM.

$$E(C|IM) = \sum_i^n E(C|d_{s,i}) \cdot P(d_{s,i}|IM) \quad (4)$$

$$var(C|IM) = \sum_i^n [var(C|d_{s,i}) + E^2(C|d_{s,i})] \cdot P(d_{s,i}|IM) - E^2(C|IM) \quad (5)$$

Finally, the damage-to-loss functions (Table.5) used in the Italian seismic risk classification framework (Cosenza et al., 2018) were adopted. Using the fragility curves presented in Figure.1, the Equations 3, 4 and 5, and the damage ratios provided in Table.5, the vulnerability curves presented in Figure.3 are obtained.

For the sake of clarity and to directly compare the obtained vulnerability curves as a function of the number of storeys, all the curves are plotted in Figure.4. Consistently with the trend highlighted through the multi-linearisation of the pushover curves presented in Mucedero et al. (2021), when increasing the number of storeys, the mean damage ratio increases.

Damage States Sismabonus	OLS	DDLs	LSLS	CLS	RLS
Damage States (this work)	DS0 (None)	DS1 Slight	DS2 Moderate	DS3 Extensive	DS4 Collapse
Damage ratio	7.0%	15%	50%	80%	100%

Table.5 Damage-to-loss function of IDR values related to damage levels assumed in this work.

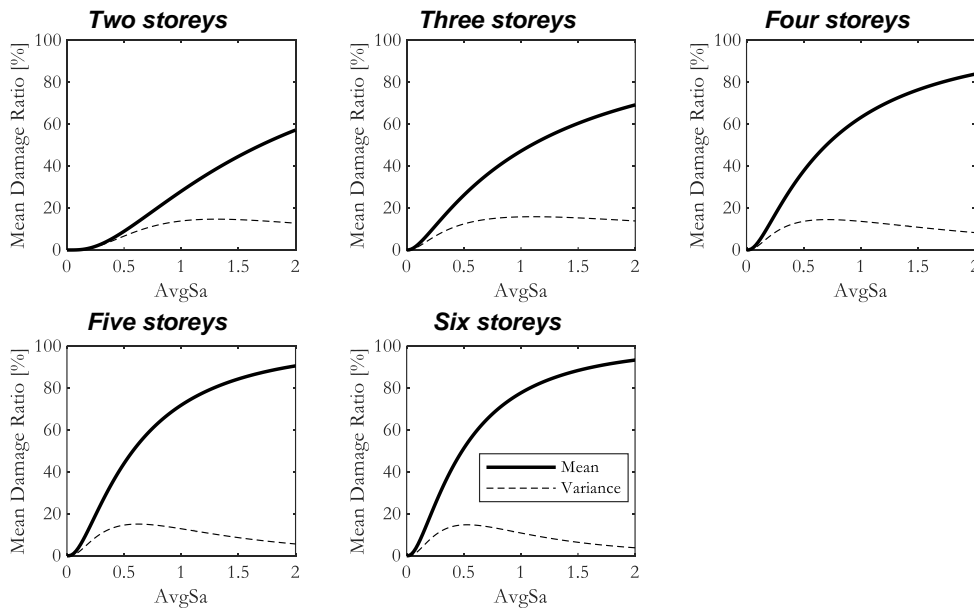


Figure.3 Vulnerability curves (mean damage ratio and variance) for RC masonry infilled buildings, as a function of the number of storeys.

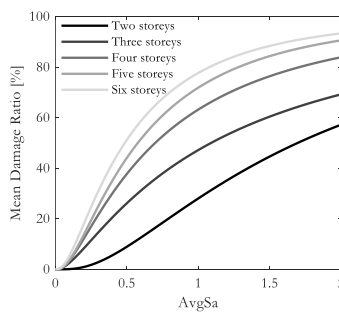


Figure.4 Comparison of vulnerability curves for RC masonry infilled buildings, as a function of the number of storeys.

CONCLUSIONS

Fragility and vulnerability curves for masonry-infilled RC buildings accounting for the wide variability in the masonry infill properties and for more realistic epistemic uncertainty levels have not been proposed so far in the literature. A step forward in this direction was taken in this study, proposing updated fragility curves, for different damage states, and vulnerability curves considering also building-to-building variability. The high level of uncertainty surrounding the

masonry infill properties was accounted for by considering five masonry infill typologies, which were selected as representative of the existing masonry infill typologies used in RC residential buildings, and by means of more refined epistemic uncertainty estimates. The fragility and vulnerability curves were developed using AvgSa as intensity measure, instead of Peak Ground Acceleration or Spectral Acceleration, as available in the literature. The obtained fragility curves show that, by increasing the number of stories, the probability to reach a given damage state increases. Such a trend is generally due to the design level, i.e., related to the increase of the seismic mass, which is not compensated by an equal increment in the capacity of the structure, since the columns were designed only for gravity loads. Moreover, it was shown how, for gravity load designed buildings, premature shear failure could happen for very low values of IDR and, consequently, the damage states generally used in the development of fragility functions are reached later. The same trend was also observed for the vulnerability curves. Moreover, the vulnerability curves for five- and six-storey buildings were aggregated, given that their differences were very minor. Finally, the updated fragility and vulnerability curves can be used for a more refined seismic risk characterisation of large building portfolios, supporting multiple stakeholders and decision-makers in quantifying and then reducing the overall seismic risk of the built assets.

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REFERENCES

- Blasi G, Perrone D, Aiello MA (2018) Fragility functions and floor spectra of RC masonry infilled frames: influence of mechanical properties of masonry infills. *Bulletin of Earthquake Engineering* 16:6105–6130. <https://doi.org/10.1007/s10518-018-0435-4>
- Boorea D, Atkinson G (2008) Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%Damped PSA at Spectral Periods between 0.01 s and 10.0 s. *Earthquake Spectra - EARTHQ SPECTRA* 24:. <https://doi.org/10.1193/1.2830434>
- Bosi A, Marazzi F, Pinto A, Tsionis G (2011) The L'Aquila (Italy) earthquake of 6 April 2009: report and analysis from a field mission (formerly at JRC).
- Braga F, Manfredi V, Masi A, et al (2011) Performance of non-structural elements in RC buildings during the L'Aquila, 2009 earthquake. *Bulletin of Earthquake Engineering* 9:307–324. <https://doi.org/10.1007/s10518-010-9205-7>
- Cardone D, Perrone G (2015) Developing fragility curves and loss functions for masonry infill walls. *Earthquake and Structures* 9:257–279. <https://doi.org/10.12989/eas.2015.9.1.257>
- Da Porto F., Donà M., Rosti A. et al. Comparative analysis of the fragility curves for Italian residential masonry and RC buildings. *Bull Earthquake Eng* 19, 3209–3252 (2021). <https://doi.org/10.1007/s10518-021-01120-1>
- De Luca F, Woods GED, Galasso C, D'Ayala D (2018) RC infilled building performance against the evidence of the 2016 EEFIT Central Italy post-earthquake reconnaissance mission: empirical fragilities and comparison with the FAST method. *Bulletin of Earthquake Engineering* 16:2943–2969. <https://doi.org/10.1007/s10518-017-0289-1>
- De Risi MT, Del Gaudio C, Ricci P, Verderame GM (2018) In-plane behaviour and damage assessment of masonry infills with hollow clay bricks in RC frames. *Eng Struct* 168:257–275. <https://doi.org/10.1016/j.engstruct.2018.04.065>
- D.M. 30/05/1972 (1972) Norme tecniche alle quali devono uniformarsi le costruzioni in conglomerato cementizio, normale e precompresso ed a struttura metallica (in Italian).
- FEMA (2012) Multi - hazard Loss Estimation Methodology Earthquake Model Hazus®-MH 2.1 Technical Manual. Washington, DC
- FEMA 356 (1997) Prestandard and Commentary for the Seismic Rehabilitation of Building.

- Furtado A, Rodrigues H, Arêde A et al (2020) Mechanical properties characterization of different types of masonry infill walls. *Front Struct Civ Eng* 14:411–434. <https://doi.org/10.1007/s11709-019-0602-y>
- Giardini D, Woessner J, Danciu L (2014) Mapping Europe ' s Seismic Hazard. *Eos Trans* 95
- Ghobarah A (2004) On drift limits associated with diferent damage levels. Performance-based seismic design concepts and implementation, vol 28. Proceedings of the international workshop, Bled, pp 321–332
- Hak S, Morandi P, Magenes G, Sullivan TJ (2012) Damage control for clay masonry infills in the design of RC frame structures. *Journal of Earthquake Engineering* 16:1–35. <https://doi.org/10.1080/13632469.2012.670575>
- Kohrangi M, Vamvatsikos D, Bazzurro P (2017) A Record Selection Methodology for Vulnerability Functions Consistent with Regional Seismic Hazard for Classes of Buildings. 16th World Conference on Earthquake Engineering
- Manfredi, V., Masi, A., Nicodemo, G. et al. Seismic fragility curves for the Italian RC residential buildings based on non-linear dynamic analyses. *Bull Earthquake Eng* 21, 2173–2214 (2023). <https://doi.org/10.1007/s10518-022-01605-7>
- Masi A, Digrisolo A, Santarsiero G (2014) Concrete strength variability in Italian RC buildings: Analysis of a large database of core tests. *Applied Mechanics and Materials* 597:283–290. <https://doi.org/10.4028/www.scientific.net/AMM.597.283>
- Masi A, Manfredi V, Cetraro G (2016) In-plane performance of RC infilled frames under seismic actions: experimental versus code provision values. In: Modena C, da Porto F, Valluzzi MR (eds) *Brick and block masonry—trends, innovations and challenges*. Taylor & Francis Group, London, pp 1245–1252. <https://doi.org/10.1201/b21889-168> (ISBN 978-1-138-02999-6)
- Morandi P, Hak S, Magenes G (2018) Performance-based interpretation of in-plane cyclic tests on RC frames with strong masonry infills. *Eng Struct* 156:503–521. <https://doi.org/10.1016/j.engstruct.2017.11.058>
- Mucedero G, Perrone D, Monteiro R (2021) Nonlinear static characterisation of masonry infilled RC building portfolios accounting for variability of infill properties. *Bulletin of Earthquake Engineering* 19:2597–2641. <https://doi.org/10.1007/s10518-021-01068-2>
- Mucedero G, Perrone D, Brunesi E, Monteiro R (2020) Numerical modelling and validation of the response of masonry infilled rc frames using experimental testing results. *Buildings* 10:1–30. <https://doi.org/10.3390/buildings10100182>
- Mucedero G, Perrone D, Monteiro R (2022) Epistemic uncertainty in poorly detailed existing frames accounting for masonry infill variability and RC shear failure. *Earthq Eng Struct Dyn*. <https://doi.org/10.1002/eqe.3748>
- Olsson A, Sandberg G, Dahlblom O (2003) On Latin hypercube sampling for structural reliability analysis. *Structural Safety* 25:47–68. [https://doi.org/10.1016/S0167-4730\(02\)00039-5](https://doi.org/10.1016/S0167-4730(02)00039-5)
- O'Reilly GJ, Sullivan TJ (2018b) Quantification of modelling uncertainty in existing Italian RC frames. *Earthq Eng Struct Dyn* 47:1054–1074. <https://doi.org/10.1002/eqe.3005>
- Porter K, Kennedy R, Bachman R (2007) Creating fragility functions for performance-based earthquake engineering. *Earthq Spectra* 23(2):471–489
- Romão X., Pereira N., Castro J. M., De Maio F., Crowley H., Silva V., Martins L. European Building Vulnerability Data Repository (Version v1.1) [Data set]. Zenodo. <http://doi.org/10.5281/zenodo.4087810>, 2020.
- SEAOC (1995) *Vision 2000 - A Framework for Performance-based Design*. Structural Engineers Association of California, California Office of Emergency Services, California, USA.
- Silva V, Crowley H, Pagani M, et al (2014) Development of the OpenQuake engine, the Global Earthquake Model's open-source software for seismic risk assessment. *Natural Hazards* 72:. <https://doi.org/10.1007/s11069-013-0618-x>
- Verderame GM, Ricci P, Esposito M, Sansiviero FC (2011) *Le Caratteristiche Meccaniche degli Acciai Impiegati nelle Strutture in C.A. Realizzate dal 1950 al 1980*. Aicap- Reluis 1–8