

IMPLICATIONS OF MASONRY INFILL-RELATED VARIABILITY ON THE SEISMIC RISK ASSESSMENT OF EXISTING RC BUILDINGS

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Abstract: *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 building collapse fragility curves and consequence models. This study deals with the derivation of fragility and vulnerability curves of existing reinforced concrete frames, considering different structural and geometrical configurations. To do so, a portfolio representative of buildings designed according to the Italian codes in force between 1970 and 1980, was used as a case study. Building-to-building variability is included through different geometrical configurations, considering buildings with different heights and in-plan layouts. Infilled and pilotis buildings were considered as common structural configurations typically found in Italy, while the infill-to-infill variability is accounted for using a macro-level distinction of five common infill types, defined in terms of stiffness and shear strength. The results of multiple-stripe analyses were used to develop fragility curves for all the different structural and geometrical configurations investigated, allowing to account for the masonry infill-related variability on the seismic collapse assessment of existing RC buildings. Finally, vulnerability curves were derived using the damage-to-loss model calibrated according to post-earthquake observation damage data from the 2009 L'Aquila earthquake and the actual repair costs monitored during the reconstruction process.*

1 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 demonstrate 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 for both RC members and masonry infills is of paramount importance within the seismic assessment of RC frame buildings, 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 account 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 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 for existing gravity load designed (GLD) infilled and pilotis buildings, accounting for the wide variability of the masonry infill properties and a more realistic epistemic uncertainty level. 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. Compared 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 at single-building and building portfolio levels.

2 Masonry infill uncertainty quantification and impact on expected annual losses

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 is 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 more 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 validated through comparison with 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 were investigated in a few research studies, however, some aspects were still left excluded. Recently, O'Reilly and Sullivan (2018), using the Correlated Latin Hypercube Sampling method, proposed by Olsson et al. (2003), 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. Their study added some aspects that were unaddressed by previous research studies, namely: (i) a more refined modelling technique (three struts) 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).

In addition to the effects of the aforementioned uncertainty on the epistemic uncertainty, Mucedero et al. (2023) showed, using the same case-study building portfolio adopted herein, how the masonry infill properties variability propagates to the estimation of the expected annual loss (EAL). In this regard, Figure 1a highlights the impact of masonry-infilled-related uncertainty on the expected annual losses for infilled buildings, as a function of the number of storeys. In terms of overall trends, for one-storey buildings, when going from infill type 1 (weakest) to infill type 5 (strongest), the losses are practically unaffected, despite a slight reduction. In turn, when increasing the number of storeys from one to four, the EALs increase with increasing stiffness and strength of the masonry infills, whilst for more than four storeys, a reduction in the EALs is verified. To emphasise the importance of properly identifying the properties of masonry infills or taking their variability into account for large-scale risk assessment studies, the CoV was used (Figure 1b) to evaluate the dispersion

around the EAL values. Different CoV ranges were identified: (i) relatively low CoVs, in the range of [0÷30%], for buildings with three or more storeys and infill type-1 (weakest); (ii) higher CoVs, in the range of [50÷65%], for buildings with six storeys and medium-to-strong (type-3 and -4) and strong (type-5) masonry infills; (iii) in the remaining cases, the CoV is in the range of [29÷49.5%]. A mean CoV was derived (around 40%) aggregating all the masonry infills for each building height, which can be useful for large-scale seismic risk assessment applications when the masonry infill properties are totally or partially unknown.

With such considerations in mind and considering the inherent difficulties of retrieving details of mechanical properties of masonry infills when conducting large-scale risk assessment studies, it becomes of paramount importance to improve fragility and vulnerability curves for existing infilled and pilotis buildings in Italy, considering the masonry infill-related uncertainty in a more thorough manner.

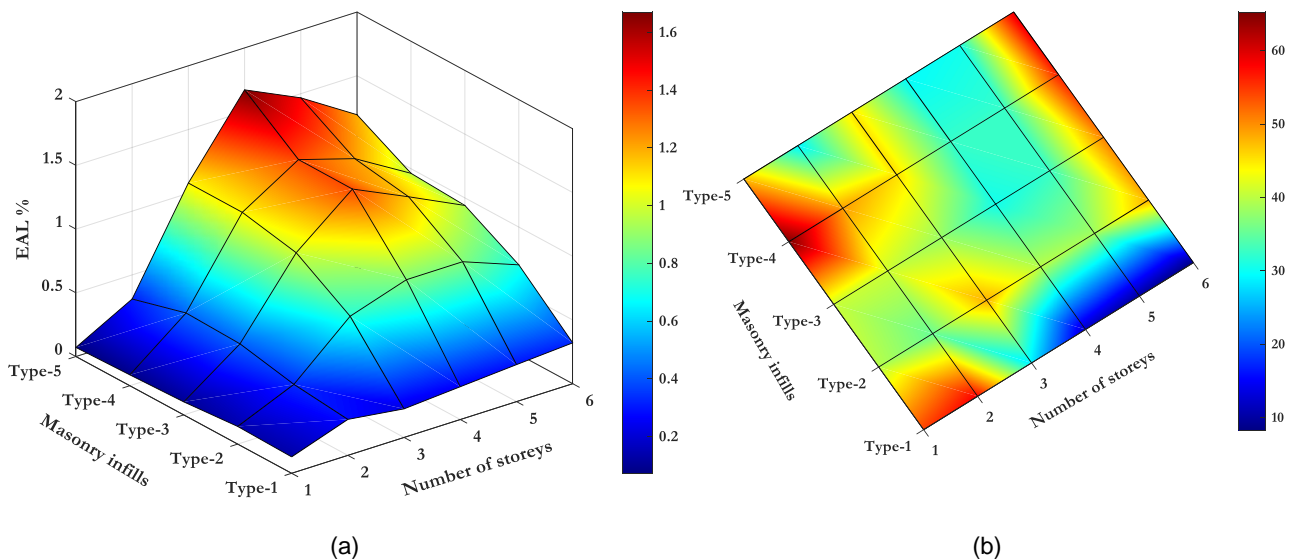


Figure 1. (a) Median Expected Annual Losses (EAL) and (b) CoV (in percentage) of the EAL as a function of the number of storeys and masonry infill type (adopted from Mucedero et al., 2022)

3 Methodology

The points below summarise briefly the adopted research methodology, which is schematically depicted in Figure 2:

- The building portfolio developed by Mucedero et al. (2021) is used as case-study, being representative of RC GLD buildings built in Italy between 1970 and 1980;
- Characterisation of the seismic hazard for the city of L'Aquila (Italy), assumed as the case-study location, 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.

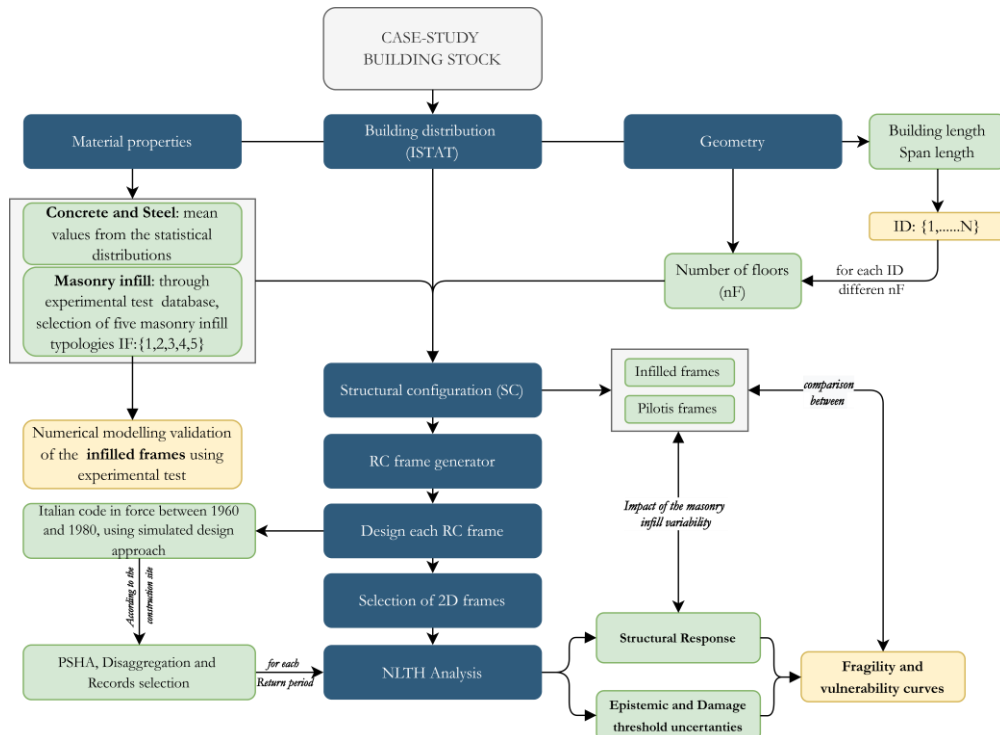


Figure 2. Adopted research methodology.

4 Case-study masonry-infilled and pilotis RC frames

4.1 Selected 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 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 by 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 by 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).

4.2 Numerical modelling approach

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. To account for these aspects, the numerical model approach proposed by Mucedero et al. (2020), whose accuracy was validated 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 relevant 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.

4.3 Seismic hazard characterization and 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 by Mucedero et al. (2021).

5 Fragility and vulnerability curves

5.1 Fragility 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 of the building portfolio, a total of 900 RC masonry-infilled buildings were analysed, of which 450 were infilled and 450 pilotis, leading to a total of 270'000 NLTH analyses.

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, numerous studies address 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 Interstorey Drift Ratio (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 1 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 2 presents the IDR thresholds assumed in this paper. 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).

Table 1. Summary of the most recent IDR thresholds for the identification of masonry infill damage states.

Reference	Statistical parameters	Damage state		
		DS1	DS2	DS3
De Risi et al. (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 et al. (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 et al. (2012)	Median [%]	0.20	0.30	1.00
Morandi et al. (2018)	Median [%]	0.30	0.50	1.75

Table 2. IDR values related to damage levels assumed in this work.

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

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 2) 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 2:

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

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, to eliminate the intersection of fragility curves related to two or more damage states, the total dispersion value (β_{TOT}) 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 Figures 3 and 4, for infilled and pilotis frames, respectively, whereas the mean values, for each damage level, and the β_{TOT} are summarised in Tables 3 and 4.

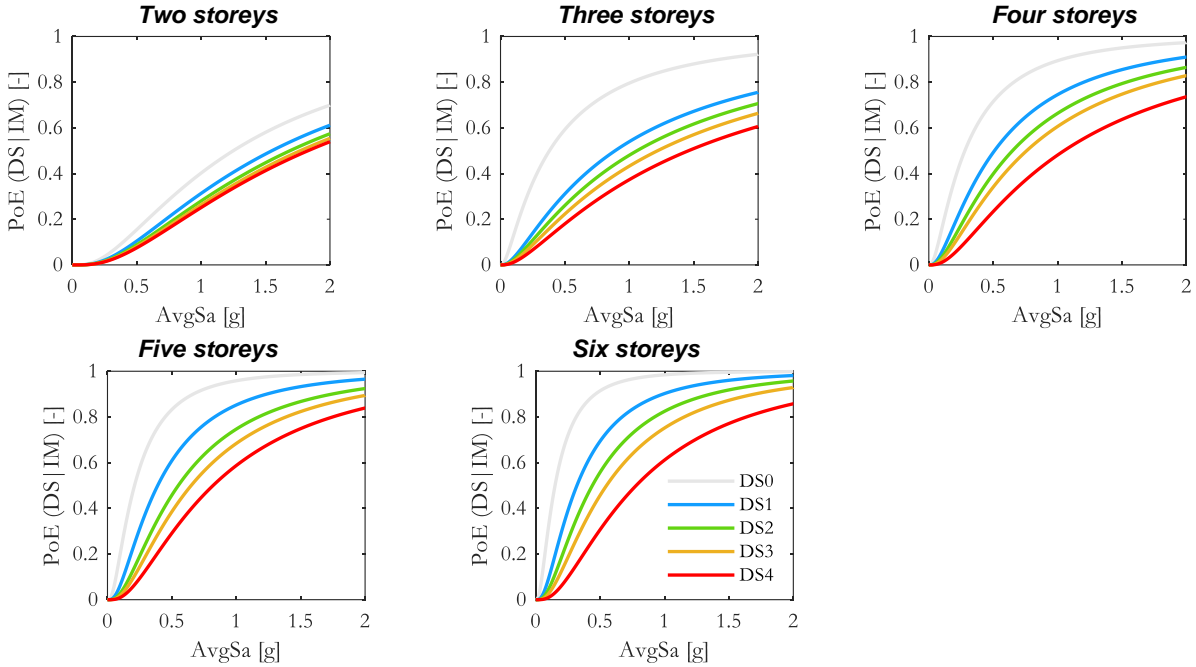


Figure 3. Fragility curves (for each damage level) for infilled buildings, as a function of the number of storeys.

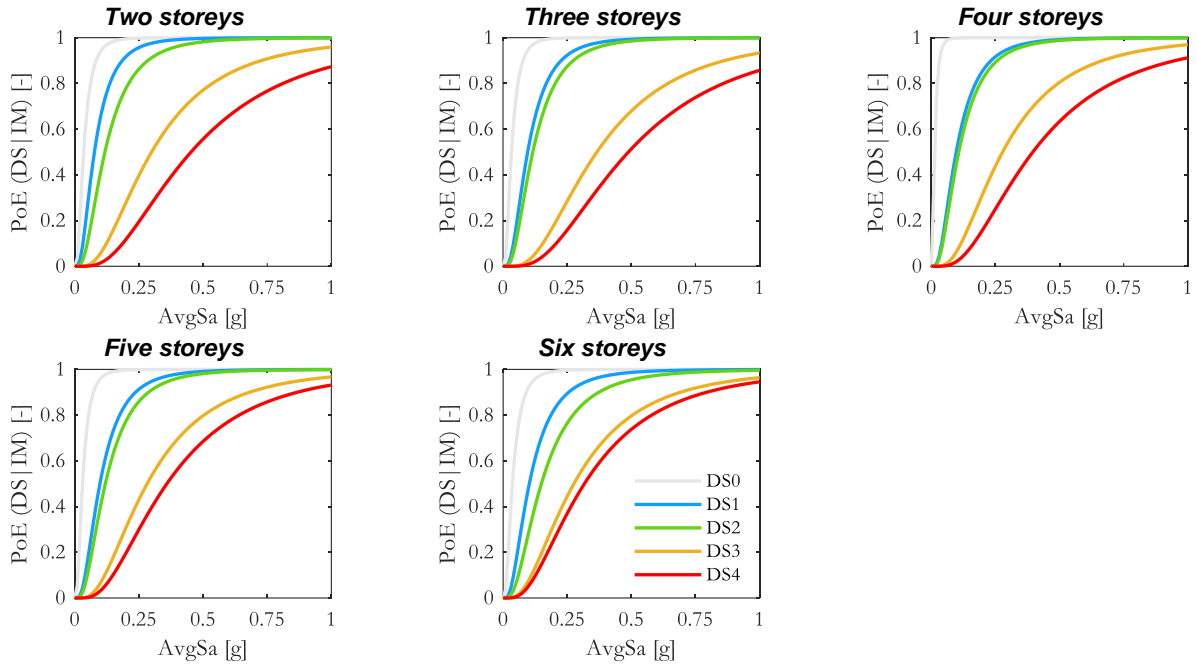


Figure 4. Fragility curves (for each damage level) for pilotis buildings, as a function of the number of storeys.

As expected, the median intensity of collapse, for each damage state, for pilotis frames is much lower than that obtained for the infilled frames. Moreover, lower overall uncertainty (β_{TOT}) is obtained for pilotis frames compared to the infilled frames, as also shown previously for both the EDPs (IDR and PFA).

Table 3. Fragility parameters for each damage state of infilled frames, as a function of the number of storeys.

Buildings	$\vartheta(DS0)$	$\vartheta(DS1)$	$\vartheta(DS2)$	$\vartheta(DS3)$	$\vartheta(DS4)$	β_{TOT}
Two storeys	1.252	1.552	1.689	1.773	1.834	0.902
Three storeys	0.379	0.894	1.057	1.216	1.464	1.174
Four storeys	0.276	0.511	0.651	0.764	1.048	1.019
Five storeys	0.207	0.387	0.547	0.655	0.824	0.905
Six storeys	0.150	0.324	0.444	0.548	0.777	0.878

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

Buildings	β (DS0)	β (DS1)	β (DS2)	β (DS3)	β (DS4)	β_{TOT}
Two storeys	0.034	0.076	0.117	0.299	0.453	0.693
Three storeys	0.033	0.096	0.116	0.366	0.489	0.670
Four storeys	0.013	0.097	0.107	0.278	0.396	0.684
Five storeys	0.030	0.098	0.119	0.282	0.359	0.688
Six storeys	0.036	0.102	0.149	0.276	0.317	0.716

5.2 Vulnerability curves

Two approaches, building-based and component-based vulnerability assessment, can be used to generate vulnerability curves. 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. The mean, $E(C > c|IM)$, and the variance, $var(C|IM)$, of the vulnerability function are then 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 Figures 3 and 4, the Equations 3, 4 and 5, and the damage ratios provided in Table 5, the vulnerability curves presented in Figures 5 and 6 are obtained.

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

Damage states Sismabonus	Operational (OLS)	Damage Limitation (DDL)	Life Safety (LSLS)	Collapse (CLS)	Reconstruction (RLS)
Damage States (this study)	DS0 (None)	DS1 Slight	DS2 Moderate	DS3 Extensive	DS4 Collapse
Damage ratio	7.0%	15%	50%	80%	100%

Consistently with the trend highlighted through the multi-linearisation of the pushover curves presented in Mucedero et al. (2021), the mean damage ratio increases for infilled frames with the number of storeys. In turn, the vulnerability curves obtained for pilotis frames are quite similar to each other for lower IM levels, regardless of the number of storeys, while becoming slightly different when increasing the IM level.

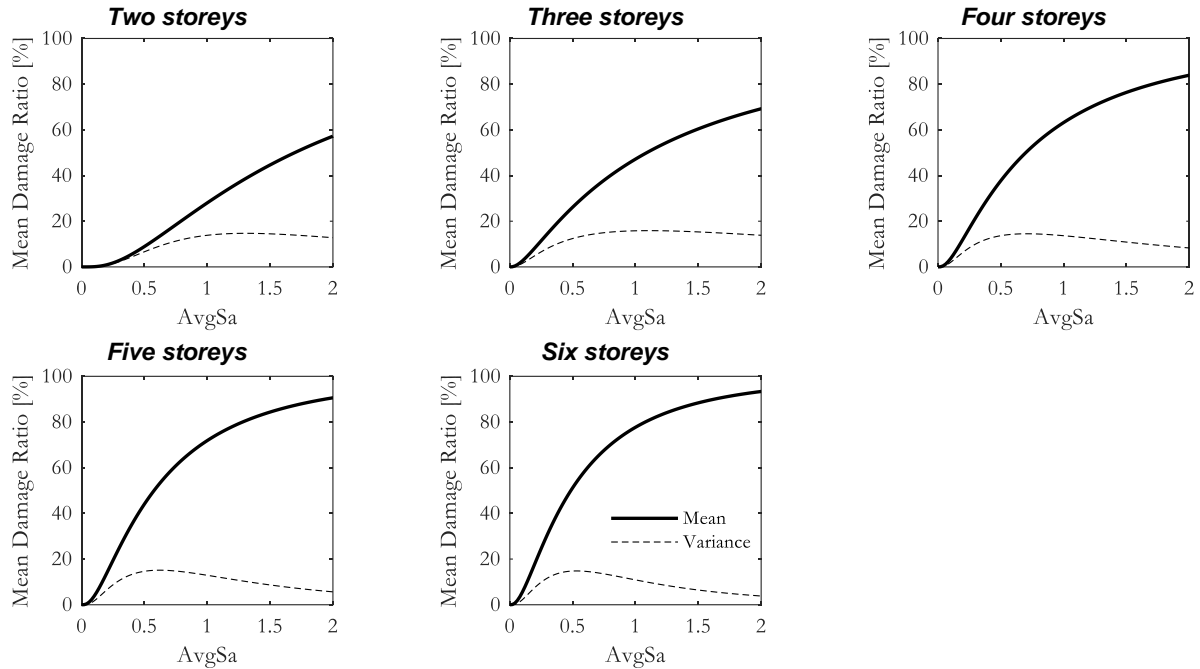


Figure 5. Vulnerability curves (mean damage ratio and variance) for infilled buildings, as a function of the number of storeys.

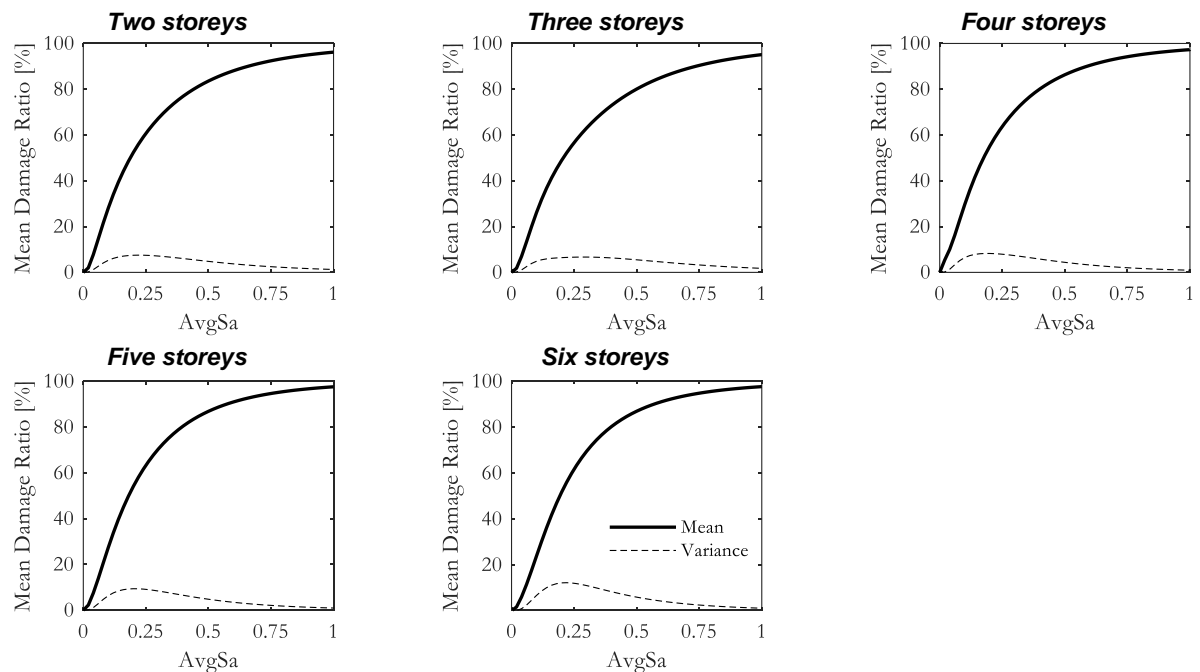


Figure 6. Vulnerability curves (mean damage ratio and variance) for pilotis buildings, as a function of the number of storeys.

6 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 using more refined epistemic uncertainty estimates. Fragility and vulnerability curves were developed using AvgSa as an 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 storeys, the probability of reaching 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. The same trend was also observed for the vulnerability curves. Pilotis frames, as expected, showed a more vulnerable behaviour compared to the infilled ones.

The advantage of using the fragility and vulnerability models developed herein lies in the fact that, in addition to the building-to-building variability, the most updated set of epistemic dispersion and the variability of the masonry infill typologies were accounted for. Thus, 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 defining mitigation strategies to reduce the overall seismic risk of the existing buildings.

7 Acknowledgements

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