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A CLASS-ORIENTED LARGE SCALE COMPARISON WITH POSTEARTHQUAKE DAMAGE FOR ABRUZZI REGION

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Abstract. *The need to quickly and accurately estimate the number of involved people, dead and the injured, missing people and homeless represent a topic issue in the emergency management and disaster planning. Obviously, such an estimation is closely related to the evaluation of building damage scenario for the area of interest. To this aim, it may be used different methodological approaches for the evaluation of seismic vulnerability at large-scale, consisting of empirical or mechanical methodologies, more or less simplified and that require a different level of detail in input parameters.*

In present work, a simplified mechanical method – PushOver on Shear Type models (POST) – for seismic vulnerability assessment of infilled RC building is briefly recalled [1, 2, 3]. The methodology allows the definition of building structural characteristic through a simulated design procedure in compliance with design code prescriptions, professional practice and seismic classification of the area of interest at the time of construction. The evaluation of non-linear static response is performed through a simplified model, considering the contribution of both RC columns and infill panels to lateral resistance, based on Shear Type assumption. Seismic capacity assessment is made in the SPO2IDA framework [4] for different performance levels, defined according to the damage classification proposed by [5]. Finally, the use of a Monte Carlo simulation approach allows the derivation of fragility curves for different damage states (DSs).

Therefore, predicted damage scenario is derived from POST methodology for a database consisting of 7597Moment Resisting Frame (MRF) residential RC buildings located in the Abruzzi region, which after the 2009 catastrophic earthquake have been charged to post-earthquake usability assessment procedure [6]. Due to the use of spatially extended and massive amount of data, the reference unit is the class of building. The choice of key parameters for the identification of classes is carried out evaluating their impact on the observed damage collected through post-earthquake survey. Then, within each class, the remaining parameters are assumed as random variables chosen in appropriate distributions evaluated from data collected during the post-earthquake survey. Correlation values among the variable of each class are evaluated. Predicted damage scenario is then compared with damage scenario based on post-earthquake data.

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1 INTRODUCTION

Italian earthquakes of the last three decades have highlighted the high seismic vulnerability of the existing building stock, as demonstrated by the significant human, financial and functionality losses due to the severe damage suffered by structural and non-structural components observed after moderate-to-high magnitude events. Therefore, there is a growing interest in seismic fragility assessment of buildings at regional and urban scale.

Generally speaking, a seismic fragility assessment can be carried out by means of analytical or empirical methods.

Analytical methods are based on the use of numerical models providing the expected damage (typically corresponding to a fixed displacement threshold) as a function of the seismic intensity. Several approaches have been developed in literature, different from each other both in structural (usually simplified) modelling and in seismic capacity assessment procedure. Among them, methods based on simplified mechanics-based procedures (e.g. Cosenza et al. [7]; Borzi et al. [8]), Capacity Spectrum Methods (e.g. Iervolino et al. [9]; Del Gaudio et al.[1,2,3]), and Displacement-Based methods (e.g. Calvi, [10]; Crowley et al. [11]; Borzi et al. [12]).

In empirical methods the assessment of the expected damage to a building is based on the observation of damage suffered during past seismic events (e.g. Braga et al. [13]; Sabetta et al. [14]; Lagomarsino and Giovinazzi [15]; Rota et al. [15]; Zuccaro and Cacace [17]). Empirical vulnerability methods have the advantage of providing a realistic damage estimation since they are derived from observed post-earthquake damage data. Hence, their reliability is related to the quality and amount of data they are based on. For instance, depending on the available building dataset, it may be necessary to convert and homogenize damage data collected in different surveys into a unique damage scale.

In present work, a simplified mechanical method – PushOver on Shear Type models (POST) – is used in order to derive seismic damage scenario for a database consisting of 7597 Moment Resisting Frame (MRF) residential RC buildings located in the Abruzzi region.

2 DATABASE DESCRIPTION

The empirical data collected during L'Aquila post-earthquake surveys are presented in this section. The 2009 L'Aquila (Abruzzi Region) earthquake hit 130 municipalities (Dolce and Goretti [18]) and caused extensive damage to public and private structures, to artistic and cultural heritage of L'Aquila and provinces.

Few days after the earthquake, the damage and usability assessment of the buildings started to determine whether they could be safely used in the case of aftershocks.

The damage and usability assessment of ordinary buildings was performed by means of the first level survey form, the AeDES form (Baggio et al. [19]).

The field inspections carried out after L'Aquila earthquake allowed to collect a database containing first-level data, on 74254 buildings (Dolce and Goretti [18]), out of which, 8750 is constituted by Moment Resisting Frame (MRF) RC buildings characterized by a residential use and located in Municipalities classified with macroseismic intensity in the Mercalli-Cancani-Sieberg (MCS) scale (Sieberg 1930) (I_{MCS}) higher than VI.

Furthermore, in order to avoid bias in the results, only buildings located in a Municipalities with a predetermined "threshold of completeness" on the survey activities are herein considered (for further details see Del Gaudio et al.[1,2,3]). According to such assumptions, the final subset consists of 7597 MRF residential RC buildings.

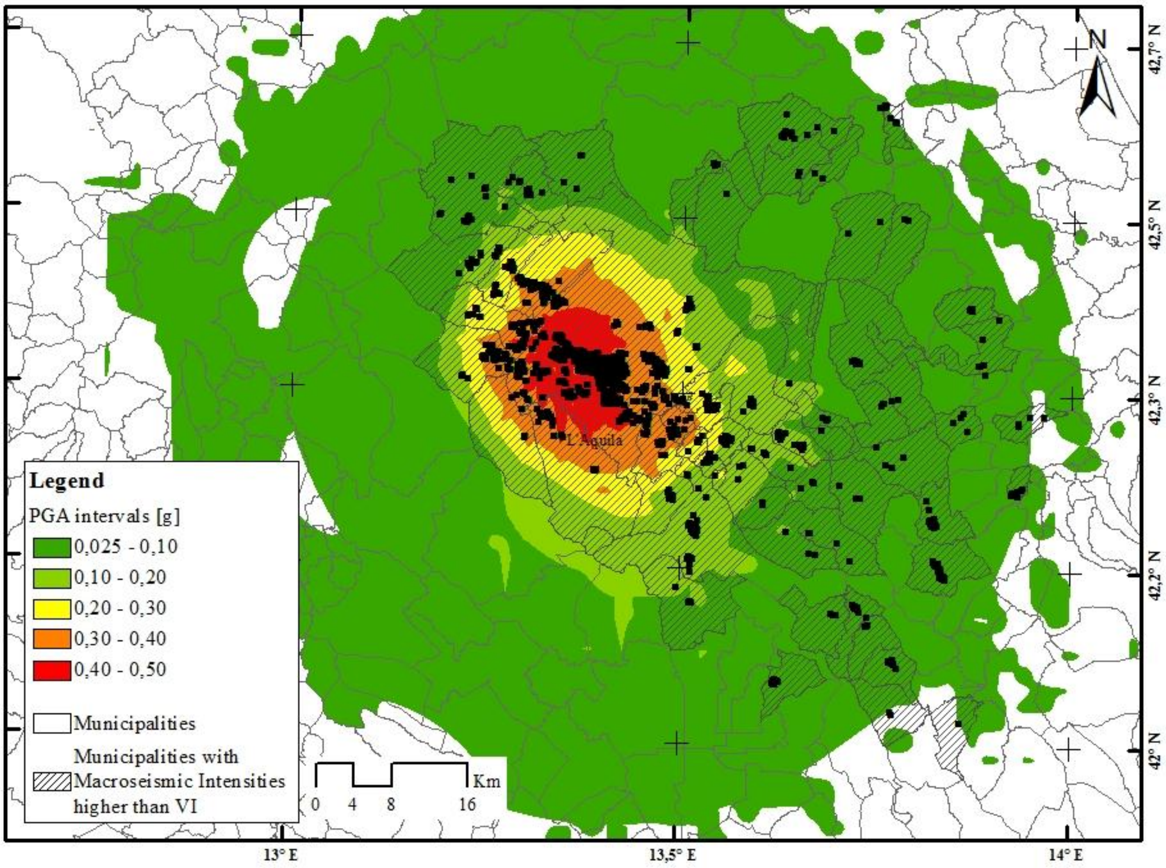
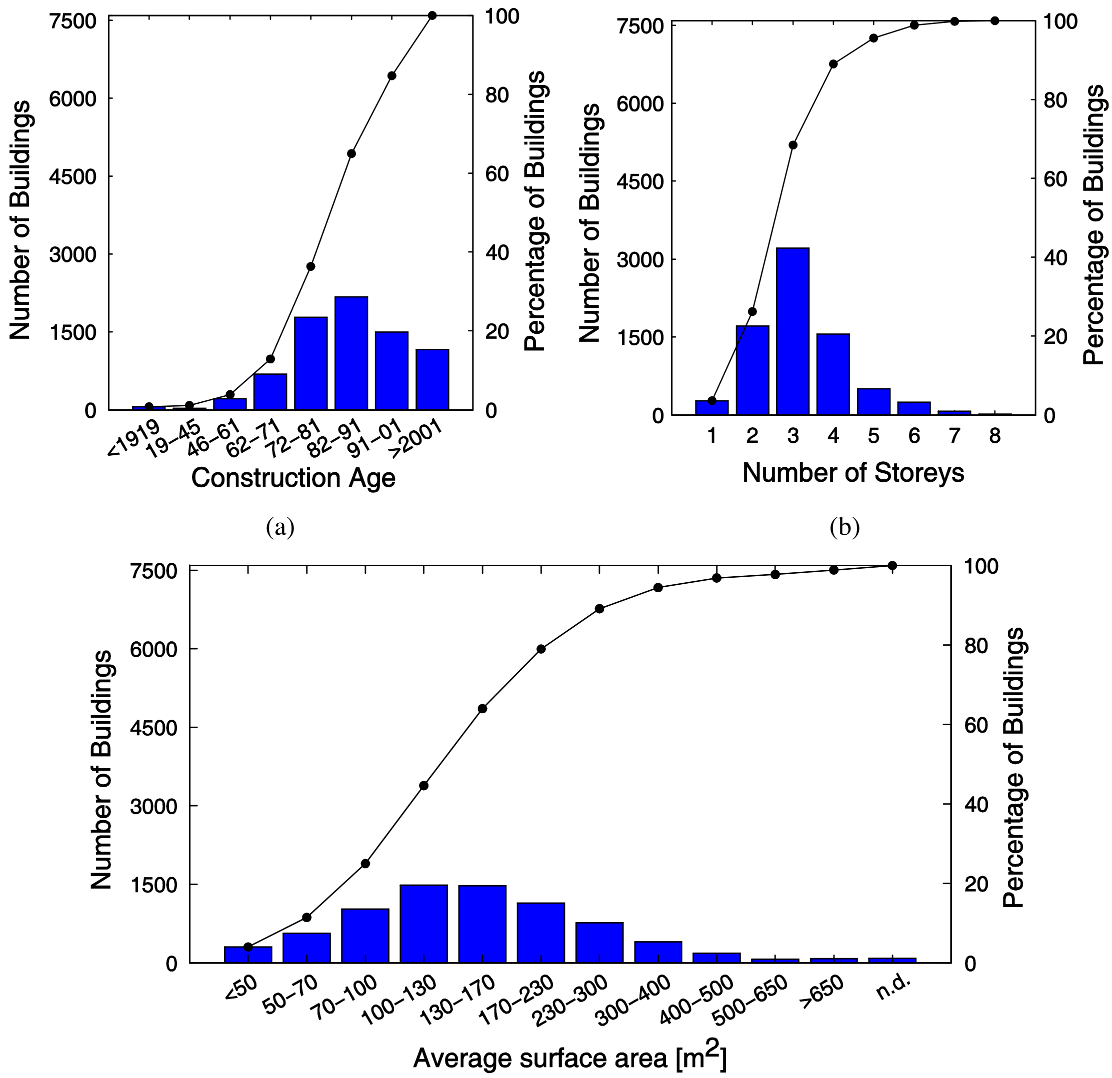


Figure 1: Isoseismic areas corresponding to the assumed PGA bins form 0.025g to 0.50g and Municipalities classified with MCS Intensity higher than VI

Figure 1 also reports the ShakeMap of the event published soon after the earthquake by the Italian National Institute of Geophysics and Volcanology (Istituto Nazionale di Geofisica e Vulcanologia, INGV), generated through the software package ShakeMap® developed by the U. S. Geological Survey Earthquake Hazards Program (Wald et al., 2006) specifically designed to obtain maps of the peak ground motion parameters (Michelini et al. [20]).

Figure 2 reports information about construction age, number of storeys and average storey surface collected during the field inspections. The construction age is classified according to eight periods (i.e. before 1919, between 1919 and 1945, 1946-1961, 1962-1971, 1972-1981, 1982-1991, 1991-2001, after 2001) as commonly adopted in the census data collections (and in the AeDES form). Figure 3 (a) shows that 87% of the building dataset (corresponding to 6616 buildings) was built after 1972; the graphs in Figure 3 (b), (c) show that 85% of the building dataset (corresponding to 6482 buildings) had a number of storeys between 2 and 4, while 67% of the building dataset (corresponding to 5131 buildings) had an average storey surface between 70m^2 and 230m^2.



(a) (b)

(c)

Figure 2: Distribution and cumulative percentage of buildings as a function of construction age (a), number of storeys (b) and average storey surface (c)

Furthermore, AeDES survey form gives information on damage (Ds) detected on structural components: vertical structures, floors, stairs, roofs and infills-partitions. The definition of the observed damage grades is based on the European Macroseismic Scale EMS98 (Grünthal [5]).

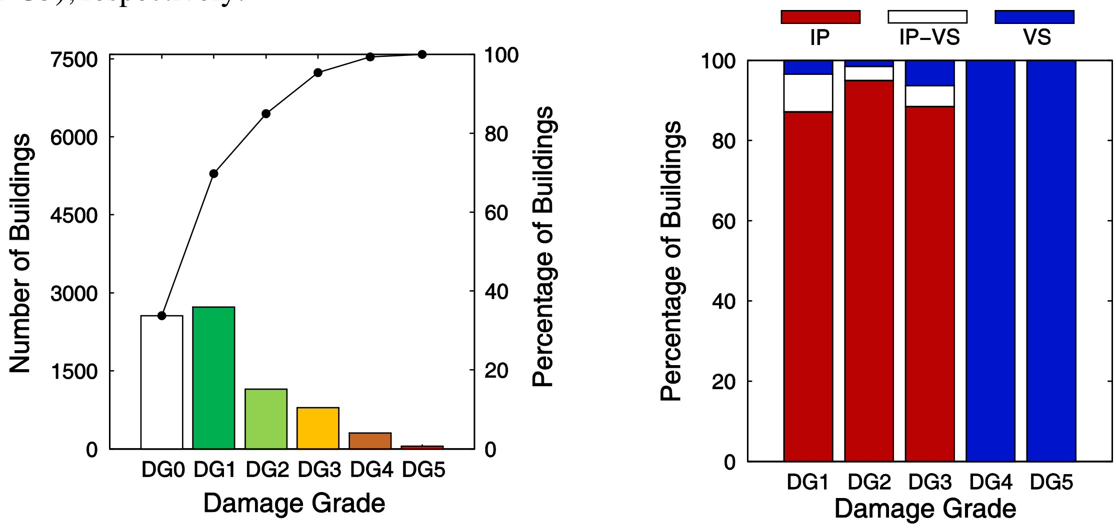
AeDES survey form provides only three Ds, combining level D2 with D3 and D4 with D5 of EMS98.

On the other hand, observed damage scenario is herein derived considering 5 damage grades (DGs) according to the EMS98 damage classification (from D0-no damage to D5-destruction). Only damage to vertical structures and infills-partitions will be used to derive observed damage scenario obtained in present work.

The equivalence between 5 (EMS98) DGs and 3 (AeDES) Ds is performed according to the scheme reported in Dolce et al [21] and Rota et al [16] for vertical structure and to the scheme reported in Del Gaudio et al [2] for infill-partitions.

Figure 3 reports the observed damage scenario for the whole building stock. It is to noted that 34% of the building stock has a “Negligible damage” (DG0), 36% ,15% and 10% belong to “Slight damage” (DG1), “Moderate damage” (DG2) and “Substantial to heavy damage”

(DG3) respectively, and only the 4% and 1% by “Very heavy damage” (DG4) and “Destruction” (DG5), respectively.



(a) (b)

Figure 3: Distribution and cumulative percentage of buildings as a function of damage grades (DGs) (a); Percentage ratio of buildings belonging to a given DG due to the maximum damage detected on VS, IP or VS-IP (b).

Figure 3b reports the percentage ratio of buildings in which the maximum damage to a given DG is detected on vertical structures (VS), infill-partitions (IP) or both (VS-IP). It is to be noted that on average, in 87% of the cases the damage to IP governs the lighter DGs of the building (DS1, DG2, DG3), highlighting the very significant role of the non-structural damage affecting infill components; vice-versa, DG4 and DG5 are related exclusively to damage to Vertical Structures (VS).

3 POST DESCRIPTION

In this section, the simplified mechanical method POST (PushOver on Shear Type models) for seismic vulnerability assessment of RC buildings is presented and briefly described; for further details, the reader is referred to (Del Gaudio et al. [1,2,3]).

The methodology is based on the following steps:

I. Definition of building model: the structural model of the building is constructed through a simulated design procedure in compliance with design code prescriptions, professional practice and the seismic classification of the area of interest at the time of construction. For further details, the reader is referred to Verderame et al [22].

II. Non-linear static response: the evaluation of the non-linear static response of the building is performed through a simplified model based on Shear Type assumption; the latter allows to consider RC columns and infill elements acting in parallel, considering a tri-linear envelope for RC columns and a multi-linear model for infill panels according to Panagiotakos and Fardis [23]. The pushover curve is obtained in closed-form through a force-controlled procedure up to the peak and through a displacement-controlled procedure thereafter. Hence the storey characterized by the maximum value of the ratio between interstorey shear demand and interstorey shear strength will be the only one to reach its maximum resistance and to experience its softening post-peak behavior, giving to the pushover curve the corresponding degrading trend.

III. Definition of DSs: DSs adopted in the proposed method are defined according to the damage scale proposed by EMS-98;

IV. Seismic capacity assessment: the SPO2IDA (Vamvatsikos and Cornell [4]) framework is applied to evaluate the seismic capacity at the performance levels of interest in terms of spectral intensity measures;

V. Evaluation of fragility curves: depending on the level of knowledge achieved, certain random variables have to be introduced. Then, a Monte Carlo simulation approach is used, generating a population of buildings. For each run of the procedure, an n-vector with the realization of each random variable, where n is the total number, is extracted from the corresponding distribution. Hence for each run the building model and the corresponding non-linear static response is determined. DSs and corresponding displacement values are determined on the building response and seismic capacity assessment is performed in terms of spectral ordinates and PGA values. Therefore, if the PGA capacity at a given DS is calculated for all the generated buildings, the corresponding cumulative frequency distributions provide the corresponding fragility curves.

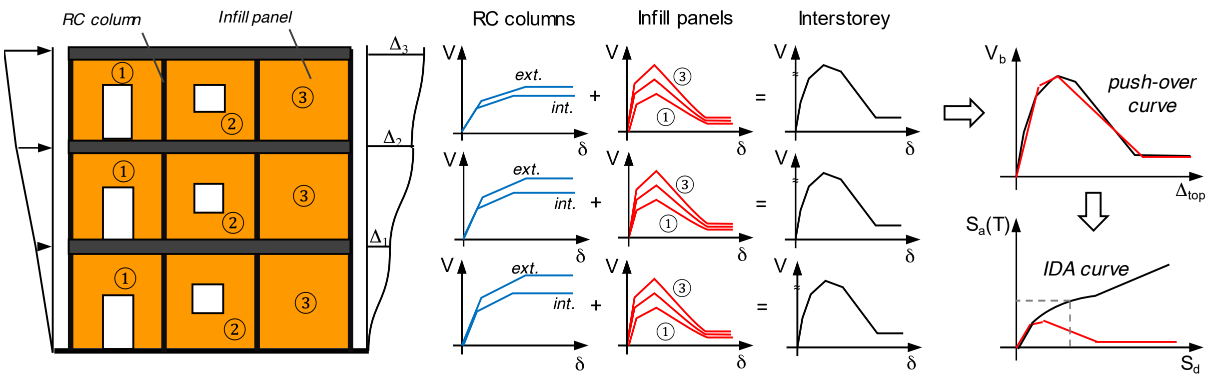


Figure 4: POST methodology: building model, non-linear static response and seismic capacity assessment

It is to worth noting that the expected failure mode is determined for each column through the comparison between the flexural plastic shear (V_y=M_y/0.5 h) and the shear strength (V_n according to Sezen and Moehle [24]). Hence, columns with (V_y/V_n)<k are expected to fail in flexure, while columns with (V_y/V_n)≥k are expected to fail in shear or flexure-shear, where the coefficient k related to the maximum ductility-related shear strength decrease, is assumed to be equal to 0.7.

In addition, different configurations for infill panels have been considered: (i) solid-, (ii) with window opening- and (iii) with door opening-panels. Openings in infill panels are considered to modify the non-linear behaviour of the infill panels according to the model presented in Kakaletsis and Karayannis [25] by means of control parameters depending on the window and door opening sizes.

A fundamental issue is represented by the definition of DSs. As a matter of fact the qualitative description of observational DSs reported by EMS-98 damage classification is translated into numerical mechanical-based displacement thresholds in the building structural model (such as the Interstorey Drift Ratio (IDR) capacity).

Capacity for infill panel ($IDR_{IP,DS1}$, $IDR_{IP,DS2}$, $IDR_{IP,DS3}$) is set according to IDR values reported in Cardone and Perrone [26].

On the other hand, the IDR capacity for RC columns is evaluated in correspondence of physical condition related to the qualitative description of EMS-98 DSs, namely to the first attainment of:

- **DS1**: the cracking moment ($IDR_{RC,cr}$);
- **DS2**: the yielding moment ($IDR_{RC,y}$);
- **DS3**: the IDR value corresponding to the concrete cover spalling and the longitudinal reinforcement buckling, ($IDR_{RC,s}$, $IDR_{RC,b}$) or the IDR value corresponding to the Shear failure according to Aslani and Miranda [27] model ($IDR_{RC,SF}$);
- **DS4**: the IDR value corresponding to the zero resistance point ($IDR_{RC,pc}$) of the degrading backbone curve provided by the model by Haselton et al. [28] or the IDR value corresponding to the loss of vertical load carrying capacity ($IDR_{RC,A-SF}$) of the first (previously shear-damaged) RC column provided by Aslani and Miranda [27];
- **DS5**: the IDR value corresponding to the zero resistance point ($IDR_{RC,pc}$) of the backbone curve provided by the model by Haselton et al. [28] for all the columns of a floor or the IDR value at which all the shear-controlled columns of a floor reach the $IDR_{RC,A-SF}$ (if both flexure- and shear-controlled columns are present in the same floor, IDR at DS5 corresponds to the attainment of zero resistance point and loss of vertical load carrying capacity in all the former and latter columns, respectively).

The numerical values of such an IDR capacity are reported in Table 2.

EMS-98 Damage State		**Infill panels**		**RC columns**		
		EMS-98 descriptions	**POST thresholds**	**EMS-98 descriptions**	**POST thresholds**	
					F-Type*	S-Type*
DS1	Negligible to slight damage	*Fine cracks in partitions and infills*	$IDR_{IP,DS1}$	*Fine cracks in plaster over frame members*	$IDR_{RC,cr}$	
DS2	Moderate damage	*Cracks in partition and infill walls.*	$IDR_{IP,DS2}$	*Cracks in columns*	$IDR_{RC,y}$	
DS3	Substantial to heavy damage	*Large cracks in partition and infill walls, failure of individual infill panels*	$IDR_{IP,DS3}$	*Spalling of concrete cover, buckling of reinforced rods*	$\min\left(IDR_{RC,s}; IDR_{RC,b}\right)$	$IDR_{RC,SF}$
DS4	Very heavy damage			*Large cracks in structural elements (...) Collapse of a few columns or of a single upper floor*	First attainment of $IDR_{RC,pc}$	First attainment of $IDR_{RC,A-SF}$
DS5	Destruction			*Collapse of ground floor or parts of buildings*	Last attainment of $IDR_{RC,pc}$	Last attainment of $IDR_{RC,A-SF}$

**: F-Type: columns failing in Flexure; S-Type: columns failing in Shear or Flexure-Shear*

Table 1: Displacement thresholds at the assumed DSs, based on the mechanical interpretation of the DSs described by EMS-98.

The resulting displacement thresholds at each DS are summarized in Table 1. Note that, due to the assumed Shear-Type behaviour, the IDR leading to the attainment of each DS is the minimum between the values reported in Table 1 for infill panels and RC columns.

The capacity in terms of spectral ordinate $S_a(T)$ is calculated through the use of an approximate IDA curve, evaluated according to the SPO2IDA framework (Vamvatsikos and Cornell, [4]) in correspondence of IDR value for the assumed DS. The corresponding PGA capacity is evaluated as a function of the adopted elastic pseudo-acceleration response spectrum.

4 DERIVATION OF CLASS-SPECIFIC ANALYTICAL FRAGILITY CURVES

POST methodology has been applied at single building level in (Del Gaudio et al. [1,2,3]).

In the present work, POST methodology will be applied at building class level. The definition of homogeneous classes of buildings will be carried out identifying the parameters that greatly influence their seismic fragility. In Del Gaudio et al. [29], two classes of buildings have been defined as a function of the *number of storeys* (N_s): "L", Low rise ($1 \leq N_s \leq 3$) buildings, and "MH", Medium-High rise ($N_s \geq 4$) buildings.

Generally speaking, some of the input parameters of the POST procedure can be modeled as random variables to reflect their expected variability. The definition of these random variables also depends on the type of analysis that is carried out. For instance, the methodology can be applied to single buildings for which the number of storeys, the plan dimensions and the age of construction can be considered as deterministically known, and other parameters are assumed as random variables, in order to account for aleatoric (e.g., the record-to-record variability, which can be taken into account, although in a simplified way, also within a spectral assessment framework as in the present study, see Section 4.1.4) or epistemic uncertainties (for parameters whose deterministic knowledge would require an excessive cost or, however, could not be easily and quickly obtained, see Section 4.1.2). If, instead, the methodology is applied to classes of buildings, i.e. in order to derive the seismic fragility of groups of buildings characterized by common values of some parameters – assumed as "distinguishing" parameters between one class and the other due to their prominent influence on seismic fragility – such as, in the present study, the number of storeys (within a given interval), further parameters have to be modeled as random variables in order to reflect their variability between one building and the other (inter-building) within the class (intra-class), such as, for instance, the number of storeys itself (within the given interval), the plan dimensions and the age of construction, see Section 4.1.1.

In this study, on the whole, $N=24$ parameters are modeled as random variables (see Table 2), each one characterized by a marginal distribution chosen to define its variability. $N = k+s+m$, with $k=1$ parameter (number of storeys) chosen to define the buildings classes, $s=3$ parameters (construction age, average storey surface and plan ratio) defining the inter-building variability within the class, and $m=20$ parameters defining the uncertainties in geometry, materials, capacity models, damage thresholds, spectral shape and record-to-record variability.

4.1 Uncertainties characterization

Below, the random variables assumed in the present work will be introduced and specialized, reporting the parameters of the distributions required for their definition (see Table 2).

4.1.1. (Intra-class) Inter-building variability

As seen previously, two classes of buildings will be introduced as a function of the *number of storeys* (N_s): *L*-class and *MH*-class of buildings.

The statistics of construction age, number of storeys and average storey surface collected during the field inspections (Figure 2) will be analyzed in order to determine the correlation matrix.

In the Monte Carlo simulation technique in the i^{th}-run, the value of construction age, number of storeys and average storey surface will be extracted so that at the end of the simulated procedure the correlation matrix between the parameters is equal to that obtained from data collected during the field inspections. The algorithm proposed by (Vorechovský and Novák, [30]) is used, which attempt to minimize the difference between the target correlation matrix and the actual correlation matrix by means of an optimal sample ordering through the so-called Simulated Annealing method.

Plan ratio is assumed as a uniform random variable between [1-2,5] according to statistics reported in (Del Gaudio et al. [1, 29]).

4.1.2. Uncertainties in geometry, materials, capacity models, damage thresholds

The selected variables are the compressive strength of concrete (f_c), steel yield strength (f_y), and infill material characteristics. The corresponding probability distributions are selected from statistical analyses provided by the technical literature to be representative of the existing Italian building stock (Del Gaudio et al. [1,2,3]).

- Compressive strength of concrete (f_c). A distribution with a mean value of 25 MPa, and a coefficient of variation (CoV) of 31% for pre-1981 buildings and of 25% thereafter, reflecting the higher reliability in the preparation of the concrete for recent construction, have been assumed.
- Steel yield strength (f_y). Distributions according to statistics reported by STIL software (Verderame et al., [31]) have been considered, with the variable as a function of the age of construction and the type of reinforcement. The latter is assumed to change with the age of construction: for buildings constructed before 1971, plain bars have been considered as reinforcement and deformed bars thereafter.
- Infill mechanical characteristics (E_w). Reference has been made to the proposal of the Italian code (Circolare 617, 2009) for hollow clay brick panels. A full correlation between the dependent variables (shear modulus G_w, shear strength τ_{cr}) and the independent variable (Young Modulus E_w) has been adopted. Hence, the elastic modulus is assumed in the following as a random variable with a lognormal distribution, a mean value equal to 4500 MPa and a CoV of 30%. G_w is evaluated as 3/10 of E_w. τ_{cr} is assumed as linearly dependent on G_w, with values of 0.3 MPa and 0.4 MPa corresponding to G_w values of 1080 MPa and 1620 MPa. The same material characteristics are assumed for external and internal infill panels.

Uncertainties in modelling are taken into account for the non-linear behaviour of both RC columns and infill panels. They are taken into consideration through a twofold contribution. On the one hand, the lateral force-displacement relationship would indirectly change through random variables related to geometrical characteristics and material properties.

On the other hand, uncertainty in modelling is explicitly considered assuming the secant stiffness at yielding (EI_y) as a random variable, determined according to model of Haselton et al. [28], for what concerning RC columns.

For infill panel, elastic and secant stiffness to peak resistance, cracking and peak resistance are considered as random variable. In the i^{th}-run of Monte Carlo simulation technique, the values will be extracted so that to minimize the difference between the actual and the target correlation matrix, the latter evaluated from the ratio between the model predictions and test result values used by the authors to derive their response model.

Furthermore, due to lack of specific information about the amount and openings in external infills, the following assumptions have been made based on expert judgement:

- Thickness of external infill panels (s_w). A discrete random variable in the interval {200; 220; 240} mm with a uniform probability distribution. In fact, the photographic documentation collected after the last 30 years earthquake has shown a thickness of infill panels equal to approximately 200-240 mm, typical of a double layer of hollow clay bricks, commonly employed in construction in Italy and in the Mediterranean area.
- The type of opening. The distribution of type of opening among the bays of a generic storey represents a critical issue that varies strongly depending on architectural aspects and professional practice. Hence, a discrete random variable is assumed with a uniform probability distribution as a function of three typologies: (i) solid-, (ii) with window opening- and (iii) with door opening-panel. The opening width is assumed to be equal to 25% of the corresponding infill length, for both window and door openings.

The distributions associated with IDR thresholds for infill panels have been identified according to the prediction model of Cardone and Perrone [26].

The distribution of IDR thresholds for RC columns (concrete cover spalling and longitudinal reinforcement buckling, post-capping IDR capacity, see Table 2) have been set according to the empirical capacity models of Haselton et al. [28] and Berry and Eberhard [32].

4.1.3. Spectral shape from ShakeMap

The spectral shape can also be modelled as a random variable.

INGV ShakeMaps provide spectral pseudo-acceleration values at T = {0; 0.3; 1.0; 3.0} s for the sites where each building is located. Hence, a Newmark-Hall elastic pseudo-acceleration spectrum can be determined, with a plateau passing through the 0.3 s acceleration, a curve passing through the 1.0 s acceleration with a 1/T decay, and another curve passing through the 3.0 s acceleration with 1/T2 decay, according to Del Gaudio et al. [3].

In such a way, 7597 elastic spectra can be evaluated from the ShakeMaps for each building site.

Hence, for each run of the Monte Carlo simulation procedure, a realization out of 7597 elastic spectra is obtained through data mining procedure previously shown.

4.1.4. Record-to-record variability (SPO2IDA relationship)

Uncertainties related to seismic ground motion are taken into account considering a lognormal distribution, whose median value is the spectral ordinate evaluated on the approximate IDA curve as function of displacement capacity. Further, logarithmic standard deviation is evaluated depending on 84%- and 16%-IDA curves. The latter is taken as equal to:

$$\beta=\frac{1}{2}\left(\ln S_a^{84\%}-\ln S_a^{16\%}\right) \tag{1}$$

where $S_a^{i^{th}\%}$ represents the ith%-fractile of spectral ordinate provided by the corresponding IDA curves provided by Vamvatsikos and Cornell (2006).

Type of R.V.	R.V.	Reference	Distribution	Median value	CoV [-]
Material properties	f_c	Verderame et al. (2001); Masi and Vona (2009)	lognormal	25 MPa	31% or 25%
	f_y	STIL Verderame et al. (2012)	lognormal	Calculated	Calculated
	E_w	Circolare 617 (2009)	lognormal	4500 MPa	30%
Geometrical characteristics	s_w	-	uniform	[200; 220; 240]	-
	Type of opening	-	uniform	[solid; window opening; door opening]	-
	Plan ratio		uniform	[1-2.5]	
	Number of storeys		uniform	Simulated annealing	
	Average surface		uniform	Simulated annealing	
	Construction age		uniform	Simulated annealing	
Modelling parameters	EI_y	Haselton et al. (2008)	lognormal	0.95*Calculated	28%
	$k_{1,infill}$		uniform	Simulated annealing	
	$k_{2,infill}$		uniform	Simulated annealing	
	F_{cr}		uniform	Simulated annealing	
	F_{max}		uniform	Simulated annealing	
Displacement thresholds	$IDR_{RC,b}$	Berry and Eberhard (2003)	lognormal	Calculated	26.2%
	$IDR_{RC,s}$	Berry and Eberhard (2003).	lognormal	Calculated	35.6%
	$IDR_{RC,cap,pl}$	Haselton et al. (2008	lognormal	1.02*Calculated	54%
	$IDR_{RC,SF}$	Aslani and Miranda (2005)	lognormal	1.05*Calculated	55%
	$IDR_{RC,A-SF}$	Aslani and Miranda (2005)	lognormal	1.03*Calculated	33%
	$IDR_{IP,DS1}$	Cardone and Perrone (2015)	normal	0.03%	59.9%
	$IDR_{IP,DS2}$	Cardone and Perrone (2015)	normal	0.35%	96.5%
	$IDR_{IP,DS3}$	Cardone and Perrone (2015)	normal	1.62%	23.7%
Spectral Shape	Spectral shape from ShakeMap	http://shakemap.rm.ingv.it	uniform	From data collected	
Record-to-record variability	Record-to-record variability	Vamvatsikos and Cornell, (2006)	lognormal	Calculated	Eq.1

Table 2: Summary of median and CoV values for the selected random variables.

5 COMPARISON IN TERMS OF FRAGILITY CURVES AND DAMAGE SCENARIO

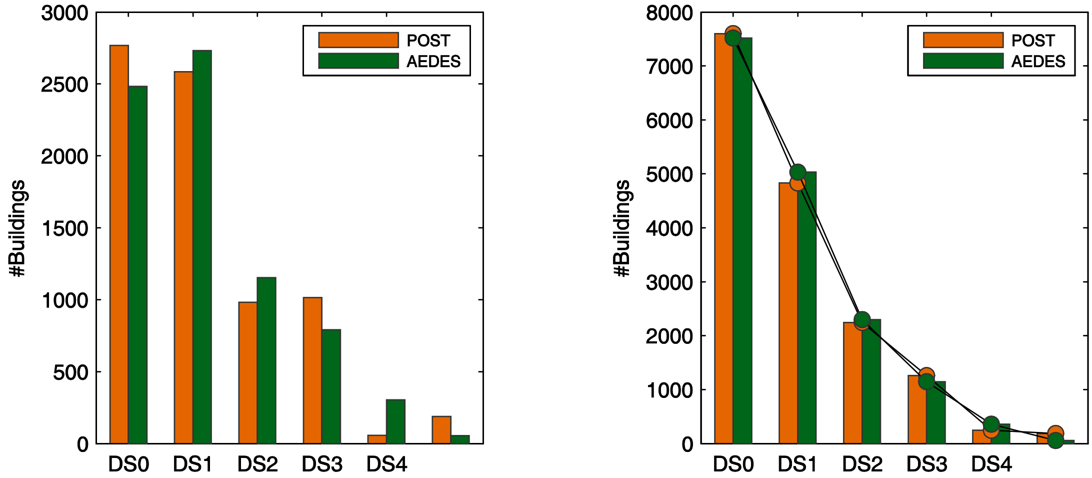
The damage scenario is evaluated based on the PGA value obtained for each building from the ShakeMap of the event.

The information about number of storeys allows to define the class of building, which the single building belongs to, and hence the corresponding fragility curves. Therefore, the latter, together with the information about the ground motion to which each building has been subjected during the 6th April 2009 seismic event (PGA value), allows to determine the probabilities of being in each assumed DS ($\Pr[ds=DS_i \mid PGA]$) for each single building.

Summing up all the corresponding probabilities of being in each DS for a N-number of buildings, the damage distribution of the considered dataset is evaluated.

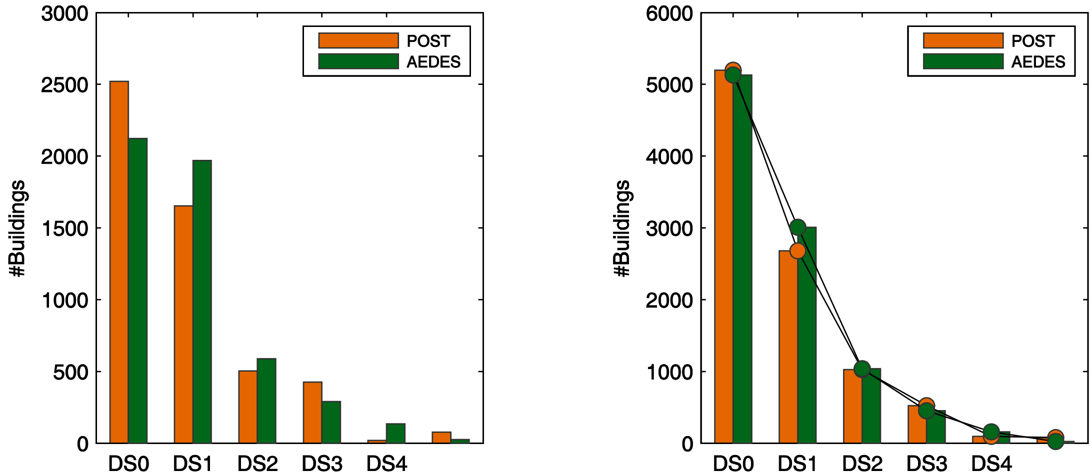
The above procedure is herein applied starting from fragility curves derived with POST methodology for L-class of buildings ($1 \leq Ns \leq 3$) and the MH-class of buildings ($Ns \geq 4$) is shown.

Hence, the distribution and cumulative distribution of damage, reporting, respectively, the number of buildings in each DSi, i=0:5, ($N[ds = DS_i]$) and the number of buildings with a damage at least equal to DSi, ($N[ds \geq DS_i]$), are shown in Figures 5, 6 and 7.



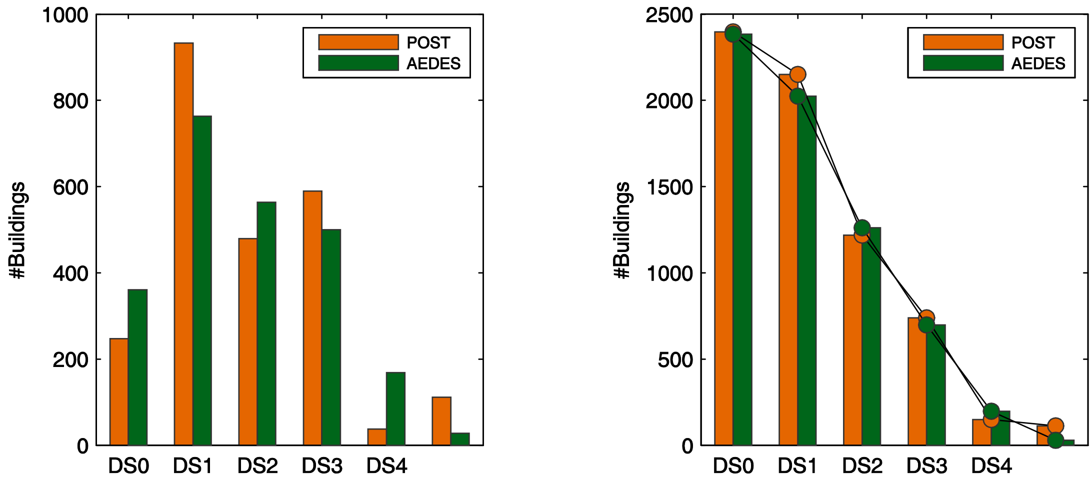
(a) (b)

Figure 5: Distribution (a) and cumulative distribution (b) of damage predicted by POST methodology



(a) (b)

Figure 6: Cumulative distribution of damage predicted by POST methodology for L-class of buildings



(a) (b)

Figure 7: Cumulative distribution of damage predicted by POST methodology for MH-class of buildings

A very good agreement from the comparison between POST-predicted and observed damage scenario can be observed. The latter derived according to section 2 from the interpretation of AeDES survey forms and already reported in figure 3a.

Furthermore, figure 6 and 7 report the same comparison for the homogenous building class above defined: L-class buildings ($1 \leq N_s \leq 3$), and MH-class buildings ($N_s \geq 4$). The good agreement between the observed and predicted results is confirmed for both building classes.

Finally, figure 8 reports the comparison between the fragility curves above discussed, derived according to POST methodology for the 7597 MRF residential RC buildings considered in the present study, with the empirical fragility curves derived on the same building dataset obtained in Del Gaudio et al., [29]. A very good agreement is observed except for DG1, where empirical curve shows a lower fragility respect to analytical POST-derived fragility curve.

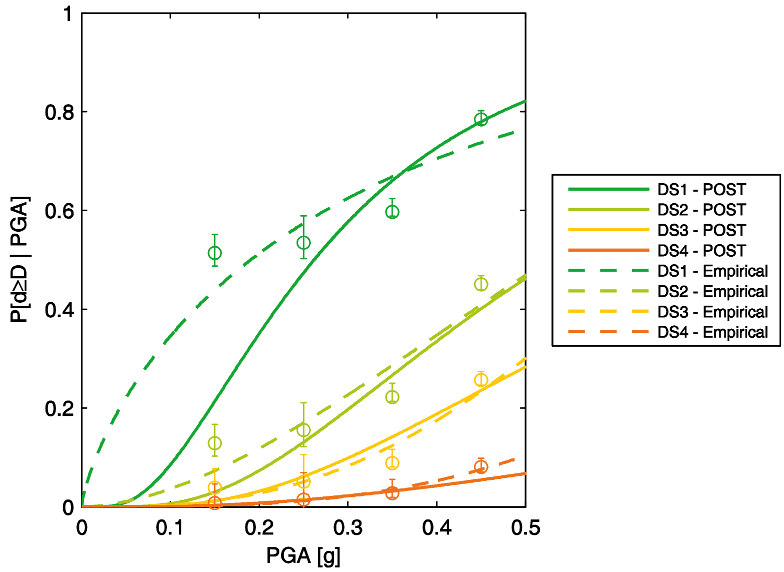


Figure 8: Comparison between empirical fragility curves for RC buildings (dashed lines) and by POST methodology (solid lines).

6 CONCLUSIONS

In present work, damage scenario is derived according to POST (PushOver on Shear Type models) methodology (Del Gaudio et al., [1, 2, 3]) for a database consisting of 7597 Moment Resisting Frame (MRF) residential RC buildings located in the Abruzzi region. Each buildings have been charged to post-earthquake usability assessment procedure after the 2009 catastrophic earthquake, leading to the definition of an observed damage scenario according to the EMS98 (Grünthal [5]) damage classification.

Due to the use of spatially extended and massive amount of data, the reference unit is the class of building, which definition of has been carried out as a function of the parameters that greatly influence seismic fragility In Del Gaudio et al. [29] two classes of buildings have been defined as a function of number of storeys (N_s): L-class buildings ($1 \leq N_s \leq 3$) and MH-class buildings ($N_s \geq 4$).

Random variables for the derivation of fragility curves are chosen in appropriate distributions, evaluated from data collected during post-earthquake survey, considering also the correlation values among the variables within each class.

In this study, on the whole, $N=24$ parameters have been modeled as random variables, each one characterized by a marginal distribution chosen to define its variability. $N = k+s+m$, with *k=1* parameter (number of storeys) chosen to define the buildings classes, *s=3* parameters (construction age, average storey surface and plan ratio) defining the inter-building variability within the class, and *m=20* parameters defining the uncertainties in geometry, materials, capacity models, damage thresholds, spectral shape and record-to-record variability.

The comparison between observed and predicted damage scenario for the 7597 MRF residential RC buildings considered in the present study, shows a very good agreement between the results, also confirmed for both predefined building classes (L rise and MH rise classes).

Fragility curves derived according to POST methodology, are then compared with the empirical fragility curves derived on the same dataset (7597 MRF residential RC buildings) obtained in Del Gaudio et al., [29], showing a very good agreement for all DGs, except for DG1, where empirical curve shows a lower fragility respect to analytical POST-derived fragility curve.

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