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A SINGLE-BUILDING COMPARISON WITH OBSERVED POST-EARTHQUAKE DAMAGE: THE CASE STUDY OF L'AQUILA MUNICIPALITY

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Abstract

A damage scenario based on observational data collected in L'Aquila Municipality after the 6th April 2009 earthquake is compared with a predicted damage scenario derived from the application of a simplified analytical method for the seismic vulnerability assessment of Reinforced Concrete (RC) buildings at large scale.

The observational damage scenario is derived from a database of 131 RC buildings located in the Municipality of L'Aquila, which after the 2009 earthquake were subjected to post-earthquake usability assessment procedure. The simplified analytical approach adopted is based on the Capacity Spectrum Method (CSM) to evaluate seismic capacity at different Damage States (DSs) based on the displacement capacity of structural and non-structural elements. DSs and the corresponding displacement capacity are defined through the interpretation of the observational-based DSs provided by the European Macroseismic Scale EMS-98. Data predicted by the adopted methodology are in good agreement with the observed damage distribution.

Keywords: RC buildings; Infills; Mechanical; Seismic fragility; Damage States; Post-earthquake

1. Introduction

Among natural disasters, earthquakes represent one of the most unpredictable, lethal and devastating phenomena from the economic and social standpoints, producing effects in spread geographical areas far away from the epicentral areas. The consequences in terms of casualties and direct or indirect damage to the structures and infrastructures are a function of the degree of urbanization and the demographic level of the affected areas, as well as the quality and type of constructions, which is significantly correlated to the presence or absence of seismic codes [1]. For this reason, there is an increasing interest in the development of methodologies able to produce reliable damage scenarios in order to support the decision process within policies of disaster prevention and emergency management.

In the following, a simplified analytical method for the seismic fragility assessment of Reinforced Concrete (RC) buildings at large scale is presented [2, 3]. The proposed method is based on a simulated design procedure to define the structural model and on non-linear static analysis of a simplified structural model based on Shear-Type assumption to evaluate seismic capacity. Damage States (DSs) are defined according to the observational-based DSs provided by the European Macroseismic Scale (EMS-98) [4]. Presence of infills is considered, both taking into account their influence on the structural response and evaluating the damage to such non-structural elements.

The methodology is used for the assessment of a damage scenario for a sample of 131 RC buildings located in L'Aquila Municipality, in the neighborhood of Pettino, subjected to the 6th April 2009 earthquake. Damage data are derived from the inspection forms collected by the Italian Department of Civil Protection (Dipartimento della Protezione Civile, DPC) right after the event. Furthermore, additional data about the location and plan dimensions of buildings collected during independent field surveys allow the construction of a geo-referenced database.

The analytical damage scenario is derived taking into account uncertainties in seismic demand, material characteristics, and modelling parameters through a Monte Carlo simulation technique. Fragility curves are obtained for each building, leading to the evaluation of the expected damage through the values of the Peak Ground Acceleration (PGA) from the ShakeMap of the event provided by the Italian National Institute of Geophysics and Volcanology (Istituto Nazionale di Geofisica e Vulcanologia, INGV).

The comparison between the observed and the predicted (analytical) damage scenario provides a support to the validation of the methodology, especially for what concerns the assumed correspondence between the DS displacement thresholds and the corresponding damage observed in structural and non-structural elements (i.e., RC columns and infill panels).

2. Damage Database

The database considered in this study is made of 131 RC Moment Resisting Frame (MRF) buildings located in Pettino neighborhood in L'Aquila. Pettino area was very close to the epicenter of the mainshock event of the 2009 L'Aquila earthquake. On April 6th, in the area between the Municipalities of Colle Roio, Genzano and Collefracido, affecting also most of Central Italy, a MW=6.3 (Ml=5.9) earthquake was recorded by INGV. Right after the mainshock event, survey campaigns of the damage, aimed at building safety evaluation, were carried out through the AeDES (Agibilità e Danno nell'Emergenza Sismica, Usability and Damage in Post-Earthquake Emergency) survey form provided by DPC [5], reporting general information of the building, about its geometrical, typological and morphological characteristics, in addition to information on extent and extension of damage to vertical structures, floors, stairs, roofs and partition and infills due to the earthquake.

The definition of the observed damage levels is based on EMS-98, as reported in [5], which includes six possible DSs (from DS0-no damage to DS5-destruction) referred to the whole building. On the other hand, AeDES survey form reports 4 damage levels, D0-no damage, D1-slight damage, and combining level D2 with D3 and D4 with D5 based on the level and on the extension of structural and non-structural damage.

Statistics about geometrical, typological and morphological characteristics, as well as observed damage collected from survey forms will be shown in the following for the buildings of the database. The 131 selected buildings are all regular in plan and elevation and fully infilled according to independent field surveys. In Fig. 1 a general overview of Pettino area in the Municipality of L'Aquila is shown, together with the ShakeMap of the event.

The buildings are mainly characterized by a number of storeys between 3 and 4 (in about 65% of cases); in particular, and more than 50% of the buildings have less than 4 storeys. Furthermore, the major part of the buildings has a plan area between 200 and 300 m2 and a plan ratio lower than 2. Plan ratio is defined as the ratio between the maximum and the minimum building dimension along the two main orthogonal directions.

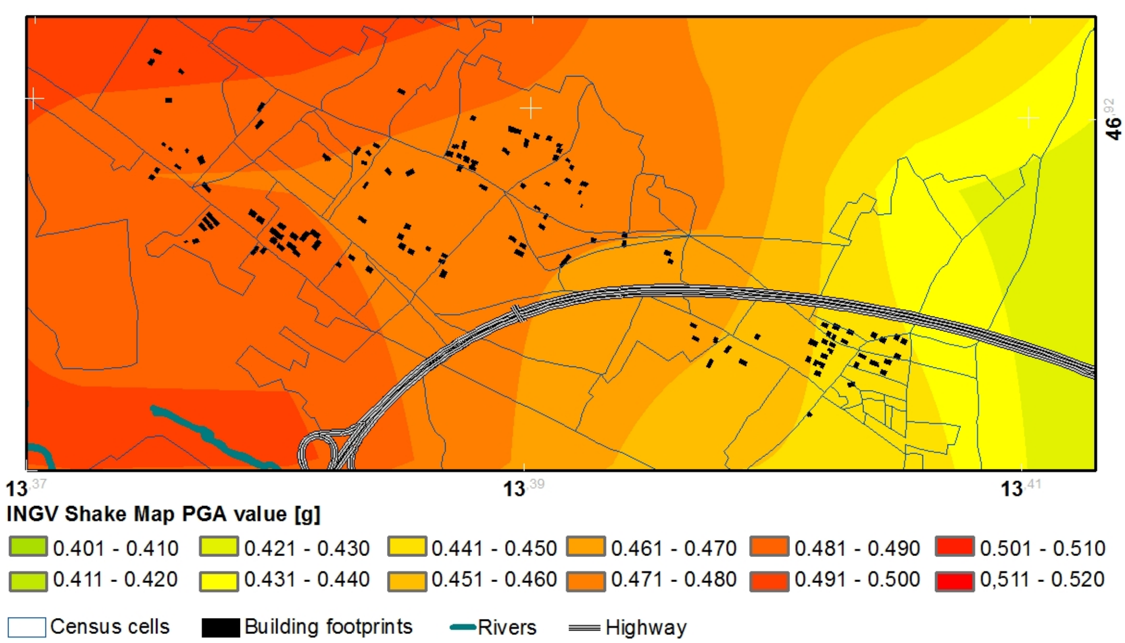


Fig. 1 – PGA data for the 6th April 2009 event according to the evaluation provided by INGV (http://ShakeMap.rm.ingv.it/shake/index.html).

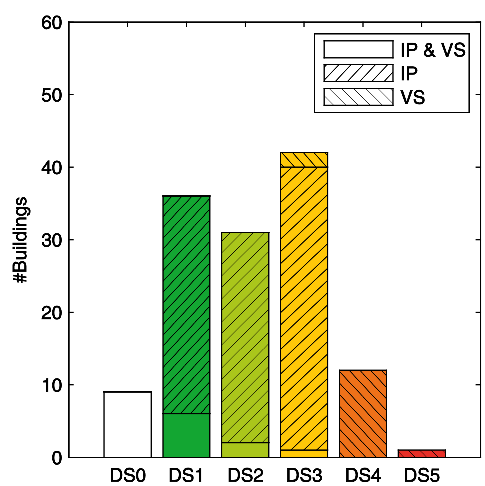
Based on the data reported in the inspection form, about 55% of the buildings is characterized by “no damage” (D0) to vertical structures, 25% by “slight damage” (D1), and only the 10% by “medium-heavy damage” (D2-D3) and “very heavy damage” (D4-D5). In addition, it can be observed that the damage is usually concentrated in a limited portion of the vertical structures, since in the vast majority of cases less than one third of the elements is damaged (D<1/3). On the other hand, a more severe and widespread damage to infill panels can be observed. As a matter of fact, only 6% of the buildings is characterized by “no damage” (D0) to infill panels and a percentage between 35 and 40% by “slight damage” (D1), “medium-heavy damage” (D2-D3) and “very heavy damage” (D4-D5).

In order to derive a damage scenario an equivalence between the adopted damage scale (i.e., EMS-98) and damage information reported in AeDES survey forms has to be set (see Fig. 2(a)). The definition of EMS-98 DSs starting from the damage to vertical structures is carried out according to the scheme reported in [6]. Therefore, for each building, a different DS for infills and vertical structures can be obtained. The heaviest DS between the two represents the DS for the whole building.

In Fig. 2(b) DS outcomes for the 131 buildings are reported. It is to be noted that most of buildings is subject to a damage between DS1 and DS3 (83%), while only a small percentage in DS0 (7%) and DS4 (9%) and a negligible percentage in DS5 (1%). Note that, according to their definition, DS4 and DS5 are related exclusively to damage to vertical structures (see Fig. 2(a)). On the other hand, DS1, DS2 and DS3 derive from damage to infill panels, thus highlighting the significant influence of non-structural damage. In particular, as far as these DSs are concerned, in about 90% of cases the damage to infill panels is more severe than that to vertical structures, whereas in about 8% of cases they exhibit the same damage and only in 2% of cases the damage to vertical structures is more severe than to infill panels.

EMS-98	AeDES Inspection form	
	Infills	Vertical structures
DS1	D1 <⅓ D1 ⅓-⅔ D1 >⅔	D1 <⅓ D1 ⅓-⅔ D1 >⅔
DS2	D2-D3 <⅓ D2-D3 ⅓-⅔ D2-D3 >⅔	D2-D3 <⅓
DS3	D4-D5<⅓ D4-D5⅓-⅔ D4-D5>⅔	D2-D3 ⅓-⅔ D2-D3 >⅔
DS4		D4-D5<⅓ D4-D5⅓-⅔
DS5		D4-D5>⅔

(a)



(b)

Fig. 2 – Assumed equivalence between EMS-98 DSs and damage levels described in AeDES survey form [5] (a) and Distribution of DSs within the database due to damage to vertical structures (VS), infill panels (IP) or both (IP and VS).

3. Methodologies for Seismic Vulnerability Assessment of RC Buildings at Large Scale

In this Section, main mechanical- and empirical-based methodologies for seismic vulnerability assessment of RC buildings at large scale from literature are briefly recalled. Comprehensive and detailed review of existing methodologies and discussion on general issues for their development can be found elsewhere [7-10].

Several mechanical-based methodologies are based on the simplified evaluation of the nonlinear static response of the building (i.e., of the buildings belonging to the class of interest). Among these, the approach proposed by [11], which is based on the Displacement-Based method, and provides the expected proportion of buildings reaching (or exceeding) the limit state displacement capacity under a given seismic intensity represented by a displacement response spectrum, taking into account a possible variation for the displacement capacity and for the corresponding secant period. The development of this procedure led to the Displacement-Based Earthquake Loss Assessment (DBELA) procedure [12].

In [13] the building class capacity is evaluated assuming geometrical and mechanical characteristics of the buildings as random variables, and generating the numerical models of single buildings through a simulated design procedure and a static pushover analysis are carried out for the generated buildings. Using a Response Surface Method, seismic risk is computed considering the number of buildings for which the displacement capacity is exceeded by the displacement demand, which is evaluated according to the Capacity Spectrum Method (CSM).

The Simplified Pushover-Based Earthquake Loss Assessment (SP-BELA) by [14] combines the definition of a pushover curve using a simplified mechanics-based procedure with a displacement-based approach, similar to DBELA. Uncertainties in geometrical dimensions, material properties, design loads, and seismic demand are taken into account. SP-BELA has been developed in order to account for the presence of infill panels [15], modelling the increase in the lateral resistance of the building up to the yield limit state due to the presence of the panels.

In [6] fragility curves derived from data on structural damage from a large database of about 150000 buildings collected after different Italian earthquakes (Irpinia 1980, Abruzzo 1984, Molise 1997, Pollino 1998, Molise 2002) have been derived. The outcomes of post-earthquake inspection forms are collected and processed in order to obtain the Damage Probability Matrices (DPMs) and fragility curves for typological classes typical of Italian building stock through non-linear regression.

In [16] a “macroseismic” and a “mechanical” method. In both cases, the adopted building typological classification essentially corresponds to the EMS-98 proposal have been combined. Following the macroseismic approach, vulnerability and fragility curves, respectively providing the expected (mean) damage grade for each building class and the probability of having each discrete damage grade as a function of macro-seismic intensity, are derived from the DPMs implicitly defined by EMS-98. The mechanical approach is based on CSM, employing bilinear Single Degree of Freedom (SDoF) capacity curves representative of each building class, which are derived from seismic design code lateral-force design requirements, factors like redundancies and conservatism, and the true strength of materials rather than the nominal ones. Hence, fragility curves are derived from the comparison between demand and capacity, the latter defined as a function of capacity curve.

4. Seismic Damage Scenarios for the 6th April 2009 L’Aquila Event

In literature, several studies have been focused on the analysis of structural damage observed in the locations affected by the 6th April 2009 L’Aquila earthquake. These studies are of different kinds: reports on the response of different structural typologies [17-20].

The main studies addressing damage scenarios in locations affected by the 6th April 2009 L’Aquila earthquake are briefly reported in this Section. In [21] damage data from about 70.000 inspections carried out under the coordination of DPC through the AeDES form (see Section 2 of AeDES form), aimed at usability assessment, were illustrated and analysed. The observed damage was defined depending on the damage to the vertical structures only, and the mean non-dimensional damage for building classes was evaluated as the weighted average of the damage levels within the classes. The Authors adopted the EMS macroseismic intensity as intensity measure. Then, the damage was analyzed depending on the building class and the macroseismic intensity; in particular, a decrease in vulnerability of RC buildings with increasing construction year and decreasing height was highlighted.

In [22] a study based on a field survey on approximately 500 geo-referenced RC frame buildings in suburban areas surrounding the historic city center of L’Aquila has been performed. The observed damage was translated into DSs according to a classification defined by the Authors, and the correlation of the observed DSs with different parameters was analyzed, concluding that non-structural damage, mainly consisting of cracking or failure of masonry infill walls, dominated the database damage assessments, and that the observed damage was significantly correlated with building height and with elevation irregularities. Moreover, fragility curves were derived assuming as ground-shaking intensity the PGA estimated from the ShakeMap of the event provided by INGV.

In [23] a study based on a field survey in the downtown of L'Aquila on approximately 1700 buildings has been carried out. Each building was geo-referenced, the observed damage was translated into the corresponding DS and a vulnerability class was assigned according to EMS-98 [4]. Hence, the spatial distribution of the observed damage and its correlation with the assigned vulnerability class were analysed, highlighting the effectiveness of EMS-98 in performing a macroseismic-based damage assessment.

In [24] damage scenarios for different recent Italian earthquakes, including the 2009 L'Aquila event, in terms of unusable and collapsed buildings, compared with the observed data have been proposed. The scenarios were evaluated by means of two different methodologies: (i) the SIGE procedure, used by DPC, and (ii) the SP-BELA procedure [14]. In particular, for masonry buildings SP-BELA was used in conjunction with the observational EMS-98-based method by [16], thus following a hybrid approach. A good agreement was thus highlighted between the predicted and observed number of buildings in each DS for this event.

The SP-BELA procedure was used also by [25], which applied to the L'Aquila earthquake a methodology for the evaluation of real-time damage scenarios following a two-level approach, i.e. providing two damage scenarios, at regional and local scale respectively. For the regional-scale scenario, census data were used for determining the building stock characteristics. For the local-scale scenario, the buildings located in the historic city center of L'Aquila were analyzed; to this end, data collected by [23] were used to assign to each building the structural type required to calculate the capacity with SP-BELA. For the regional-scale scenario a damage underestimation was observed, especially for the most severe damage grade. On the contrary, for the local-scale scenario a damage overestimation was generally observed.

In [26], based on the same damage database used in this study, DSs were derived according to EMS-98 depending on the damage affecting non-structural elements (i.e., infill panels), and these damage data, together with the PGA estimated from the ShakeMap of the event provided by INGV, allowed deriving observational fragility curves. In the same study, the simplified spectral-based mechanical procedure FAST [27] was used to derive analytical fragility curves at the same DSs. A calibration was performed on selected input parameters of FAST procedure in order to optimize the agreement between observational and analytical fragility curves. This calibration provided optimizing values in good agreement with typical values assumed in literature for the same parameters.

5. Simplified Mechanical Method for Seismic Vulnerability Assessment of RC Buildings (POST)

Hereinafter, a simplified mechanical method – PushOver on Shear Type models (POST) – for seismic vulnerability assessment of RC building is presented and briefly described; for further details the reader is referred to [2, 3]. The reference unit of the procedure is the single building. The methodology is based on the following steps: *(i)* definition of building model, *(ii)* nonlinear static response, *(iii)* seismic capacity assessment; *(iv)* definition of DSs and *(v)* Evaluation of fragility curves;

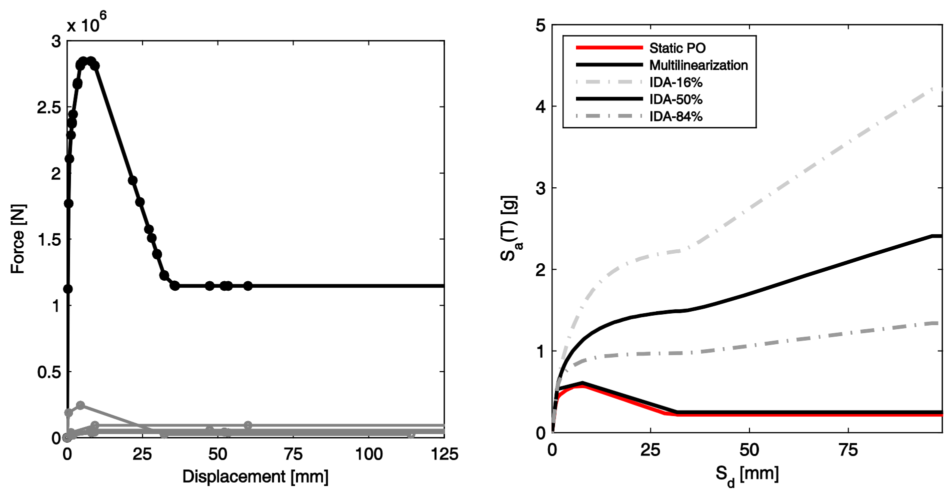
Definition of building model

The procedure is based on few geometrical data that allow to define a geometrical-structural model of the building. A simulated design of the structural model is carried out in compliance with design code prescriptions, professional practice and seismic classification of the area of interest at the time of construction. The design can be carried out for gravity loads only or for gravity and seismic loads (for further details, see [28]).

Nonlinear static response

The evaluation of the non-linear static response of the building is performed through a simplified model. It is assumed that the ends of the columns at each storey are restrained against rotation (Shear Type model). Despite the simplification, the hypothesis of Shear Type model is still able to reproduce the typical seismic response of existing RC MRF buildings with a reasonable degree of approximation, both in presence and in absence of infill. Then, the nonlinear response of RC columns and infill elements is determined. First, a tri-linear envelope is assumed for the moment-rotation model of RC column, with cracking and yielding as characteristic points. Behaviour is assumed linear elastic up to cracking and perfectly-plastic after yielding. Moment at yielding (My) is calculated in closed form by means of the first principles-based simplified formulations proposed in [29]. Rotation at yielding (θy) is univocally identified by My and the secant stiffness to yield, EIy, provided by [30]. Lateral force-displacement relationships for infill panels are evaluated according to the model proposed by [31].

The relationship between the interstorey displacement and the corresponding interstorey shear is then evaluated considering all the RC columns and infill elements acting in parallel. In this way, a multi-linear interstorey shear-displacement relationship is obtained at each storey by adding up the lateral shear-displacement relationships of all the RC columns and infill panels. Assuming a distribution of lateral forces, the shape of the corresponding distribution of interstorey shear demand can be determined. Hence, the pushover curve is obtained through a force-controlled procedure up to the peak, and by means of a displacement-controlled procedure after the peak.



(a) (b)

Fig. 3 – Example interstorey shear-displacement relationship (a) obtained as the sum of shear-displacement contributions of RC columns and infill panels, reported with grey lines, capacity curve for the Equivalent SDoF system and corresponding approximate IDA-curves obtained according to [33] (b).

Seismic capacity assessment

Once the multi-linearization of the pushover curve is carried out, the characteristic parameters of the "capacity curve" of the equivalent SDoF system are determined. Then, simplified Incremental Dynamic Analysis (IDA) curves are derived according to [33], which allows obtaining a relationship between the seismic intensity measure expressed as the spectral ordinate Sa(T), where T is the period of vibration of the equivalent SDoF, and an Engineering Demand Parameter (EDP), ductility or displacement, see Fig. 3. Then, the corresponding seismic intensity measure in terms of PGA, or any other spectral ordinate, can be derived depending on the assumed spectral shape.

Definition of Damage States

DSs adopted in the proposed methodology are defined according to the damage scale proposed by European Macroseismic Scale (EMS-98) [4]. To this aim, analytical displacement thresholds corresponding to the damage to structural and non-structural elements described by EMS-98, based on the mechanical interpretation of the description of damage reported in [4], are assumed. Hence, the qualitative description of damage provided by EMS-98 has been translated in analytical displacement thresholds through engineering judgment, separately for infill panels and RC columns, as summarized in Table 1 (for further details see [34]).

Note that, due to the assumed Shear-Type behaviour, the interstorey drift ratio, IDR, leading to the attainment of each DS is the minimum between the values reported in Table 1 for infill panels and RC columns. Once displacement capacity at each DS is determined, the corresponding spectral ordinate is evaluated from IDA curve and the corresponding PGA capacity is calculated from elastic response spectrum.

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

EMS-98 Damage States		Infill panels		RC columns	
		EMS-98 description	**POST threshold**	**EMS-98 description**	**POST threshold**
DS1	Negligible to slight damage	*Fine cracks in partitions and infills*	1st State [36]	*Fine cracks in plaster over frame members*	First attainment of cracking moment at the column end section,
DS2	Moderate damage	*Cracks in partition and infill walls.*	2nd State [36]	*Cracks in columns*	First attainment of yielding moment (or yielding chord rotation) of RC columns
DS3	Substantial to heavy damage	*Large cracks in partition and infill walls, failure of individual infill panels*	4th State [36]	*Spalling of concrete cover, buckling of reinforced rods*	First attainment ofdisplacements corresponding to Spalling of concrete cover, buckling of reinforced rods from [35]
DS4	Very heavy damage	-	-	*Large cracks in structural elements (...) Collapse of a few columns or of a single upper floor*	First attainment of the IDR corresponding to the zero resistance point of RC column backbone curve [30]
DS5	Destruction	-	-	*Collapse of ground floor or parts of buildings*	Last attainment of the IDR corresponding to the zero resistance point of RC column backbone curve [30]

Evaluation of fragility curves

The adopted methodology for the evaluation of fragility curves in a population of buildings has been described in [2]. Given a single defined building, some variables can be assumed as random variables. The distributions assumed in this work to characterize random variables related to capacity models and displacement thresholds are described hereinafter and their distributions specialized to the case study scenario in §4.1.

Uncertainty in modelling is taken into account assuming the secant stiffness at yielding in RC columns (EIy) as a random variable, according to [30]. The Authors investigate uncertainty associated with the prediction identified by the logarithmic standard deviation and by the average of the ratio between the observed and predicted values (μ=0.95; β=0.28), assuming that the model parameter follows a lognormal distribution.

The distributions associated with IDR threshold for RC elements are defined according to what reported in [30] and [35]. The distributions associated with IDR threshold for infill panels have been identified according to Table 2, where the parameters of probabilistic distributions of the drift related to certain degrees of damage to infills panels based on the pseudo-dynamic tests on infilled RC frames are reported [36].

Table 2 - Definition of IDR thresholds for infill panels from [36].

Damage State (EMS-98)	**Damage state and damage description according to [36]**	**μ [%]**	**CoV [%]**
DS1	1st: onset of cracking in the bricks, associated with the first noticeable reduction of stiffness	0.029	59.9
DS2	2nd: moderate cracks before attaining the maximum strength	0.350	96.5
DS3	4th: so many broken bricks that repair is unreasonable; reconstruction needed	1.618	23.7

Finally, record-to-record variability can be directly estimated evaluating 84% and 16% IDA curves through SPO2IDA [33], see Fig. 3.

A Monte Carlo simulation approach is used, and sampling of random variables is carried out through the efficient stratified Latin Hypercube Sampling (LHS) technique, adopting the "median" sampling scheme. In this way, a population of buildings is generated, each one corresponding to a different set of values of the defined random variables. Therefore, if PGA capacity, at a given DS, is calculated for all the generated buildings, the corresponding cumulative frequency distributions of the obtained PGA capacity values provide the fragility curves in longitudinal and transverse directions and at each DS. In the same way, fragility curves independent of the direction can be obtained, through the evaluation of the cumulative frequency distribution of the minimum PGA capacities between longitudinal and transverse direction for each sampling.

5.1 Application to Pettino buildings

In this Section, the POST procedure described above is used to derive a seismic damage scenario for the 131 buildings located in Pettino, described in §2, under the 2009 L'Aquila earthquake. In the following the main steps of the methodology (I -V) will be specialized to the present case study.

The considered buildings are symmetric in plan, both in longitudinal (X) and in transverse (Y) direction. Number of storeys, longitudinal dimension, L_x, and transverse dimension, L_y, in addition to – of course – the plan area, A_b, are available from survey data. The structural models of buildings are defined by means of a simulated design procedure [28]. Geometrical and mechanical properties of infill panels, their distribution along the height of buildings, as well as the presence and size of openings, have to be set. In this study, a uniform distribution of external infill panels both in plan and in elevation is assumed. Internal infill panels are considered, too. The thickness of external infill panels is assumed equal to 200 mm; the area of internal infill panels along each direction is assumed equal to 50% of the corresponding area of external infill panels, and their thickness is derived accordingly, based on the number of internal frames along each direction [37]. The influence of openings is taken into account through the introduction of control parameters reported in [32]. The type of opening is assumed as a discrete random variable with a uniform probability distribution, as a function of three typologies, namely (i) solid panels, (ii) panels with window opening and (iii) panels with door opening. The opening width ($L_{opening}$) is assumed equal to the 25% of the corresponding infill length (L_{inf}), both for window and door opening.

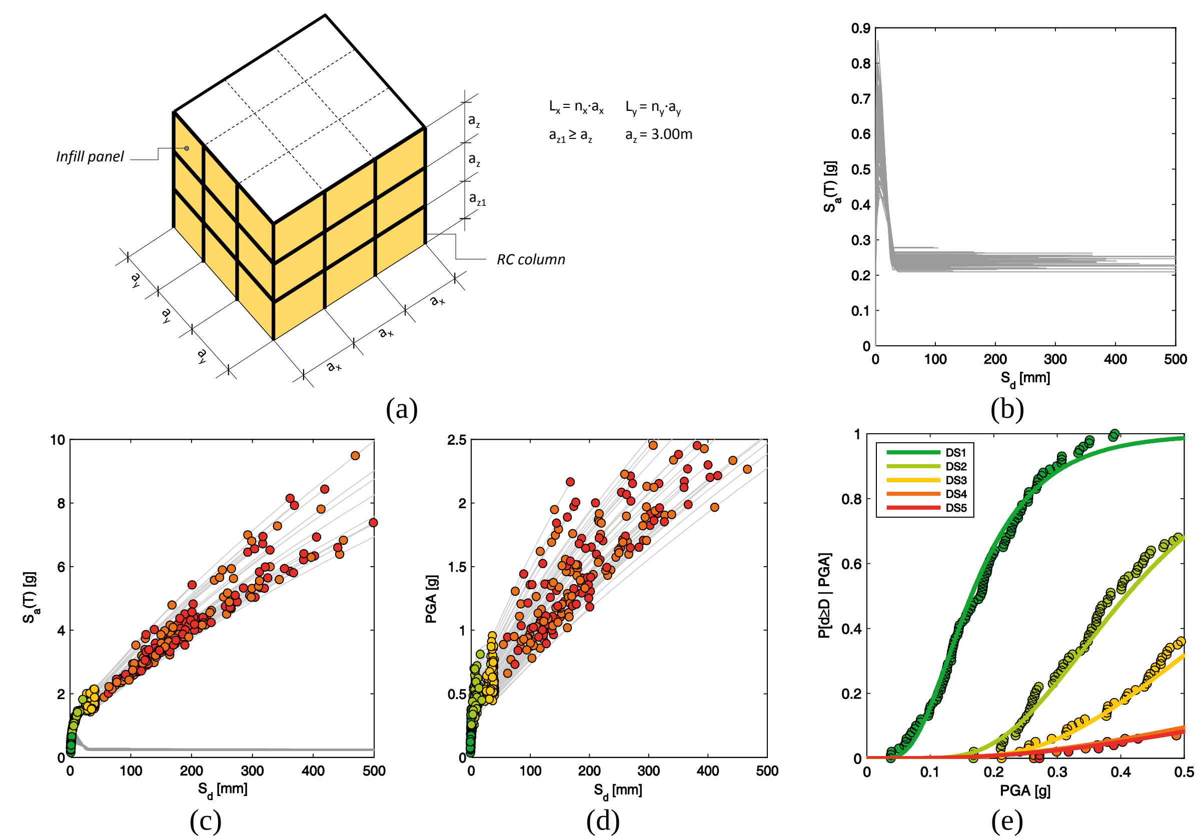


Fig. 4 – Main steps of POST methodology: model of the example 3-storey building (a), capacity curves of the equivalent SDoF systems generated through the Monte Carlo simulation technique (b), corresponding IDA curves in terms of Sa(T) (c) and PGA (d), and fragility curves at each DS in terms of PGA (e).

In the present work, the Uniform Hazard Newmark-Hall demand spectra (Type 1) provided in [38] are adopted. The spectral shape corresponding to soil type B is assumed, based on the microzonation study for L'Aquila area, edit by Gruppo di Lavoro MS–AQ (2010). However, note that this assumption affects only the spectral shape used in the derivation of the PGA capacity, at each DS, form the corresponding Sa(T) capacity.

The distributions of mechanical characteristics used in the present case study are selected in order to be representative of the existing Italian building stock. Hence, a value of 25 MPa for all ages of construction and a Coefficient of Variation (CoV) of 31% until 1981 have been assumed, while for buildings constructed after 1981 a CoV of 25% is assumed [39-40]. Mechanical characteristics of steel are evaluated through STIL software [41]. In particular, for buildings constructed before 1971 the reinforcement is assumed to be constituted by plain bars and subsequently by deformed bars. Values for infill mechanical characteristics based on the proposal of the Italian code [42] for hollow clay brick panels have been set.

In Fig. 4, the results of the main steps of POST methodology with reference to an example building from the database – characterized by Lx = 14.00m, Ly = 12.00m, Nstoreys = 3, dating back to the decade between 1972-1981 – are presented. In Fig. 4(a) the sketch of this building model is reported. The capacity curves of the equivalent SDoF systems (Fig. 4(b)) and the corresponding IDA curves expressed in terms of spectral ordinate, Sa(T), (Fig. 4(c)) and PGA (Fig. 4(d)), derived according to the Monte Carlo simulation technique, are reported. Each capacity curve, Sa(T) IDA curve, and PGA IDA curve is derived according to steps I-IV of POST methodology assuming a different set of values for the defined random variables. Finally, in Fig. 4(e) fragility curves at different DSs are reported, derived through the lognormal fit of the empirical cumulative frequency distribution of the corresponding PGA capacity values at each DS.

6. Observed and Predicted Earthquake Damage Scenarios

Damage scenario from fragility curves of Pettino buildings evaluated according to POST procedure (see §5 and §5.1) and from the ShakeMap of the event, see Fig. 1, will be derived and compared with the observed post-earthquake damage.

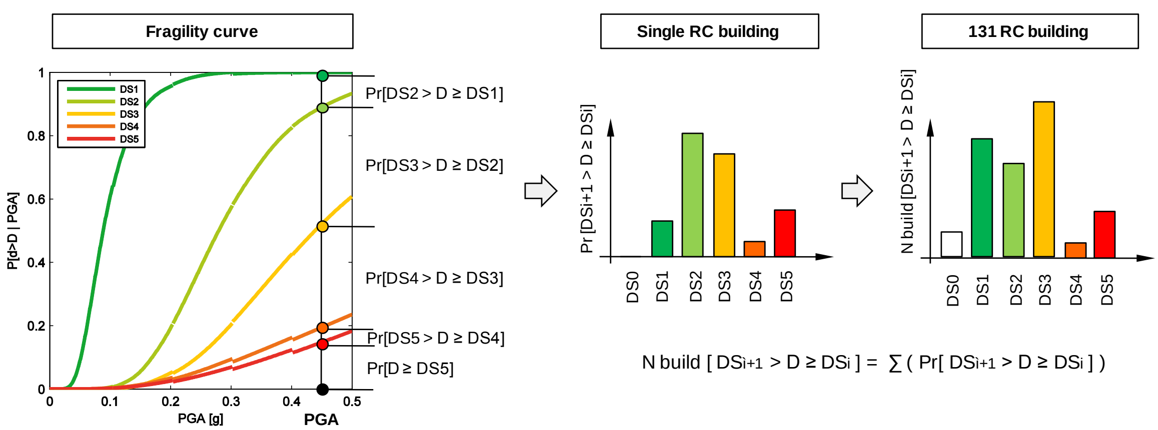


Fig. 5 – Conceptual representation of the procedure for deriving damage scenarios for the whole database starting from seismic fragility assessment of single buildings.

Damage scenarios for the whole database are derived summing up all damage distributions for the 131 buildings (see Fig. 5). The damage distribution for each single building is derived from the fragility curves at each DS and the PGA value provided by the ShakeMap of the event, due to the availability of a geo-referenced building database. Finally, damage scenarios can be compared with observed damage resulting from post-earthquake survey through inspection forms.

In Fig. 6 the distribution and cumulative distribution of damage predicted by POST methodology for the whole database are reported. A very good agreement from the comparison between predicted and observed damage scenario can be observed, the latter derived according to Section 2 of AeDES form (see Fig. 2). Nevertheless, a slight overestimation of predicted damage compared to observed damage can be noted for very heavy damage class (DS4-5).

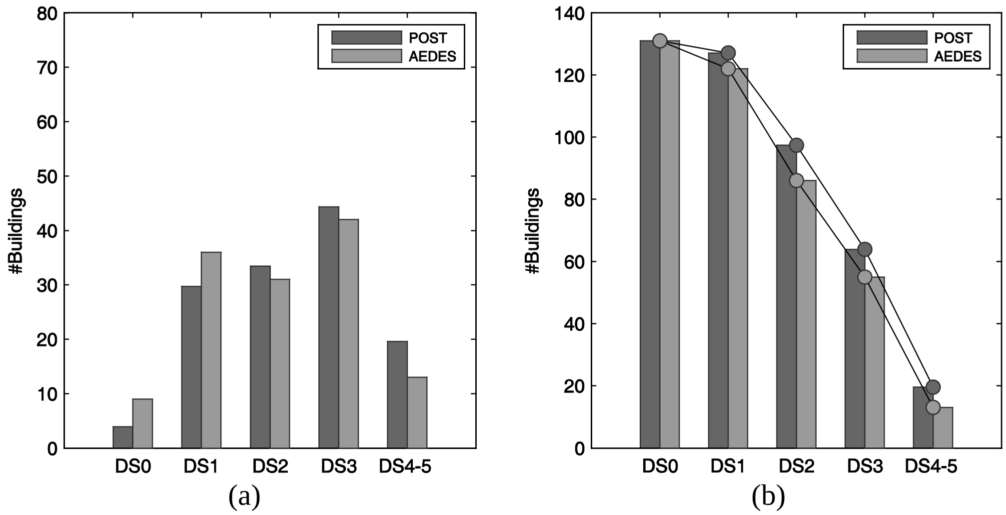


Fig. 6 – Comparison of the distribution (a) and cumulative distribution (b) of damage predicted by POST methodology with observed damage.

7. Conclusions

In this work, a comparison between the damage observed in 131 RC buildings located in Pettino, L'Aquila, subjected to the 6th April 2009 earthquake, and the damage scenario predicted by a simplified mechanical-based methodology is shown.

The POST methodology [2, 3] is based on a simulated design procedure to define structural characteristics from few building data, and employs a CSM-based procedure to evaluate the seismic capacity, assuming a shear type behaviour for the simplified structural model. Infill elements are included in the model. DSs are defined according to EMS-98, thus allowing a direct comparison with observed damage. Uncertainties in seismic demand, material characteristics, modelling parameters, and DS thresholds are taken into account through a Monte Carlo simulation technique, leading to the construction of fragility curves that allow the derivation of predicted damage scenarios starting from the site-specific seismic demand provided by the ShakeMap of the event.

The observational damage database has been described. It is derived from the survey forms filled after the event by DPC [5], integrated by additional data about the location and dimension of buildings collected during independent field surveys.

Analytical damage scenarios showed a good agreement with observed damage, both in terms of expected number of buildings in each DS and of the corresponding cumulative distribution.

Due to its nature, the methodology – which is quite low computationally demanding – has the advantage of modelling explicitly the damage to structural and non-structural elements, thus allowing a more transparent and reliable comparison with observed damage, and making it suitably applicable, for example, to component-by-component loss estimation methodologies. Further analytical-observational comparisons are foreseen in order to validate and develop the methodology from different standpoints, namely the definition of DS thresholds (which could be carried out through a hybrid calibration, too), the choice of reference intensity measure, the modelling of further uncertainties, and the development of single building or building class-based urban scale damage scenarios depending on the available level of knowledge on building stock characteristics.

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