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Fragility curves of gravity-load designed RC buildings with regularity in plan

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Abstract. In this paper Fragility Curves (FCs) relevant to existing RC framed building types representative of the Italian building population designed only to vertical load and regular in-plan have been derived from an extensive campaign of non-linear dynamic analyses. In the generation of the FCs, damage states according to the EMS98 scale have been considered while the intensity measure has been defined by adopting an integral parameter, such as the Housner intensity. FCs have been generated by varying different parameters, including building age, number of storeys, presence and position of infill panels, plan dimensions, external beams stiffness and concrete strength. In order to verify the effectiveness of the damage prediction, comparisons were made between the results obtained from the proposed FCs with those deriving from both prominent fragility studies available in the technical literature and damage distributions observed in past earthquakes. Results show that damage grades obtained by adopting the proposed FCs are generally lower than those provided by the other approaches considered. A comparison with real damage data, shows that the proposed FCs generally estimate more severe damage distributions than those observed in past earthquakes, although they give lower differences with respect to the other approaches.

Keywords: existing buildings; reinforced concrete; seismic vulnerability; fragility curves; nonlinear dynamic analyses; Housner intensity

1. Introduction

More than half of the world population currently lives in urban areas and the global rural-urban balance is increasingly in favour of cities (UNFPA 2011). Reinforced Concrete (RC) buildings represent a large portion of the built environment of urban areas in many countries all over the world, including Italy and other Mediterranean earthquake-prone countries. Existing RC buildings were also frequently designed only for gravity loads before the introduction of seismic code provisions or had outdated anti-seismic criteria, and thus lack of detailing and structural system design able to provide adequate seismic performances. As a consequence, they have often displayed unsatisfactory seismic behaviour during past earthquakes (e.g., Southern Italy 1980, Turkey 1999, L'Aquila 2009). Fatalities due to strong earthquakes are increasingly determined by To date, investigations have focused on various hybrid post-tensioned seismic structures, including frames and shear walls (Stone *et al*. 1995, Stanton *et al*. 1998). In these studies, hybrid post-

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RC building collapse (Coburn and Spence 2002). An example is provided by the 2009 earthquake which caused a total of 197 fatalities in the urban centre of L'Aquila. More than 130 fatalities (66%) were due to RC building failure, even though the percentage of RC buildings in the urban centre of L'Aquila was around 30%, with a prevalent proportion of masonry buildings (about 60%). Therefore, the assessment and reduction of RC building vulnerability are of primary concern in the mitigation of seismic risk.

Several methods are currently available for the assessment of building vulnerability. They are based on: (i) observation of damage due to past earthquakes (empirical approach), (ii) judgment of experts (expert approach), (iii) numerical analysis (analytical approach), and (iv) a combination of such approaches (hybrid approach). A comprehensive overview of the available methods can be found in Calvi *et al*. (2006).

Aimed at estimating expected damage on RC buildings, several fragility relationships were specifically defined in the past (e.g., Onose 1982, ATC 1985, Singhal *et al*. 1997, Mosalam *et al*. 1997, FEMA-NIBS 1999, Polese *et al*. 2008, Celik and Ellingwood 2009, Silva *et al*. 2013). Generally, they have been based on RC style constructions adopted in different countries of the world and, sometimes, on empirical formulae calibrated on the observed behavior and damage data from local earthquakes. As a consequence, their capability in the loss prediction of Italian and European built environment appears to be open to critique, if not dispute, unless an appropriate consideration of their structural similitude is made (Rossetto and Elnashai 2003).

Analytical fragility relationships are essentially based on the statistical treatment of seismic response results computed for different earthquake demands. Within the analytical approach framework, fragility curves (FCs) provide the probability that structural response to a given seismic ground motion would reach or exceed various states of damage (Milutinovic and Trendafiloski 2003). Analytical FCs were originally proposed in the HAZUS Technical Manual (FEMA-NIBS 1999) for U.S. buildings and lifeline systems. Afterwards, different methods of generating FCs for building types present in the European and Mediterranean region were developed. Based on current European building types, the RISK-UE project (Milutinovic and Trendafiloski 2003) developed several methods (e.g., Kappos *et al*. 2006, Lagomarsino and Giovinazzi 2006) of preparing earthquake scenarios specifically relevant to European towns, proposing FCs for various building types including RC dwelling buildings. FCs that do not comply with the U.S. building types considered in HAZUS were also generated in the framework of the EERI-PAGER (2012) initiative. Specifically, RC frame structures with and without masonry infill walls, representative of the built environment in different countries (e.g., India, Italy, Greece and Turkey), designed with and without seismic criteria, were analysed. Different analytical approaches, such as the DBELA method (Crowley *et al*. 2004) and the AUTH method (Kappos *et al*. 2006), of generating FCs were adopted and the results were compared. The set of FCs developed by Silva *et al*. (2013) for ductile and non-ductile Turkish RC buildings with and without masonry infill walls are also worthy of note.

Analytically-derived FCs are based on the definition of a given set of damage states obtained by considering mechanical response parameters and an appropriate intensity measure describing ground motion. In the original formulation by HAZUS (FEMA-NIBS 1999), FCs were described in terms of spectral displacements, $S_d(T)$, for four damage states involving both structural and non structural components. Seismic response was calculated by non-linear static analysis. Afterwards, different intensity measures and methods of evaluating the building response were adopted in developing FCs. Specifically, as described in detail in the SYNER-G (2011) report, existing FCs for RC buildings can be grouped into two main classes, based on either empirical or instrumental intensity measure types, with respect to the different types of intensity measure typically adopted.. For the former, macro-seismic intensity scales such as MCS (Sieberg 1930) and EMS98 (Grünthal 1998) were generally used, while, for the latter, seismic input refers to peak (e.g., PGA, PGV) or spectral (e.g., $S_a(T)$, $S_d(T)$) intensity measures. Furthermore, other studies focused their attention on integral intensity measures, such as Arias Intensity I_A and Housner Intensity I_H, showing their higher capability of representing the damage potential of ground motions (e.g., Masi 2003, Masi *et al*. 2011). Finally, regarding the methods adopted for the evaluation of seismic response, analytical FCs can be grouped into two main classes, FCs derived from either non-linear static-based approaches (e.g., Polese *et al*. 2008, the UTCB method given in RISK-UE (Milutinovic and Trendafiloski 2003); the AUTH method given in PAGER (D'Ayala *et al*. 2012)) or non-linear dynamic-based ones (e.g., Jeong and Elnashai 2006; the IZIIS approach given in RISK-UE).

An examination of the approaches so far adopted in obtaining the FCs for RC buildings shows that some issues require further consideration. Firstly, FCs need to be defined by adopting models which are representative of real structures typically present in the built environment under examination, and thus the results provided in approaches such as HAZUS cannot be directly adopted for European buildings. A first step towards overcoming this shortcoming was taken in the RISK-UE Project. However, the recognition of the fundamental role of seismic input in evaluating seismic response (e.g., Kwon and Elnashai 2006) suggests that the results achieved in the RISK-UE Project, which are generally based on non-linear static analyses, should be sustained by more realistic non linear dynamic analyses. Secondly, existing RC buildings frequently have a framed structure. The seismic behaviour of such structural types is significantly influenced by masonry infills, especially when building design has taken into account only vertical loads. Thus, the results obtained by applying the IZIIS approach given in RISK-UE, although based on non-linear dynamic analyses, need to be extended because they are relevant to RC framed buildings where the contribution of masonry infills was neglected.

The present paper, starts from an extensive campaign of non-linear dynamic analyses carried out by Masi and Vona (2012), to generate FCs of existing RC building types largely present in the Italian and European countries using a HAZUS-like approach. In the definition of FCs, a set of five damage states according to the EMS98 scale was considered, while the intensity measure was defined by adopting an integral parameter such as the Housner intensity (I_H). The results deriving from the FCs proposed in the present study were analysed and compared with those provided by other prominent approaches present in the technical literature and, finally, with damage data from past earthquakes.

2. Evaluation of seismic capacity

The FCs proposed in the present paper derive from a wide parametric analysis carried out by Masi and Vona (2012), in which the seismic vulnerability assessment of some existing Italian RC building types designed only to vertical loads was carried out. In accordance with a consolidate procedure firstly proposed in (Masi 2003), the methodology is made up of the following main steps:

1. selection of building types;
2. simulated design of the selected building types;
3. modelling of the building types including masonry infills;
4. execution of non-linear dynamic analyses;

5. definition of the damage scale;
6. generation of FCs.

After a brief description of steps 1-4, already extensively described in Masi and Vona (2012), the damage scale was defined (step 5) and, finally, the FCs of the selected building types were obtained in section 3.

2.1 Selection and simulated design of building types

Selection of building types is firstly based on data collected through the survey of population and building stock carried out over the entire Italian territory by the National Institute of Statistics (ISTAT census, www.istat.it). A great many buildings without significant plan irregularity can be found in the built environment, while irregularity in elevation is frequently found. Therefore, typical buildings with plan regularity have been considered in the present study. They are characterized by symmetric plan shape having either small or large floor area, that is 3 or 5 bays along the *X* direction (longer direction), respectively (Fig. 1(a)). Two bays along the *Y* direction (shorter direction) are always considered. Bay length is 5 m for both *X* and *Y* direction. In elevation, the selected types have 2, 4 and 8 storeys representative of low-, mid- and high-rise buildings according to the classification given in RISK-UE (Milutinovic and Trendafiloski 2003), with constant inter-storey height equal to 3.0 m (Fig. 1(b)). The structures have lateral load resisting frames only along the *X* direction, while along the transversal direction *Y*, in accordance with usual design practice, beams are present only in the exterior frames. With regard to the beam stiffness of the exterior frames, rigid beams (30×50 cm, RB) or flexible beams (70×22 cm, FB) were considered. Furthermore, the presence and position of infill masonry walls within the external frames were also considered (Fig. 1(c)), thus obtaining Bare Frame types (BF, frames without effective infills, i.e., with infills having many and/or very large openings or badly connected to the structure so that their contribution to the strength and stiffness of the

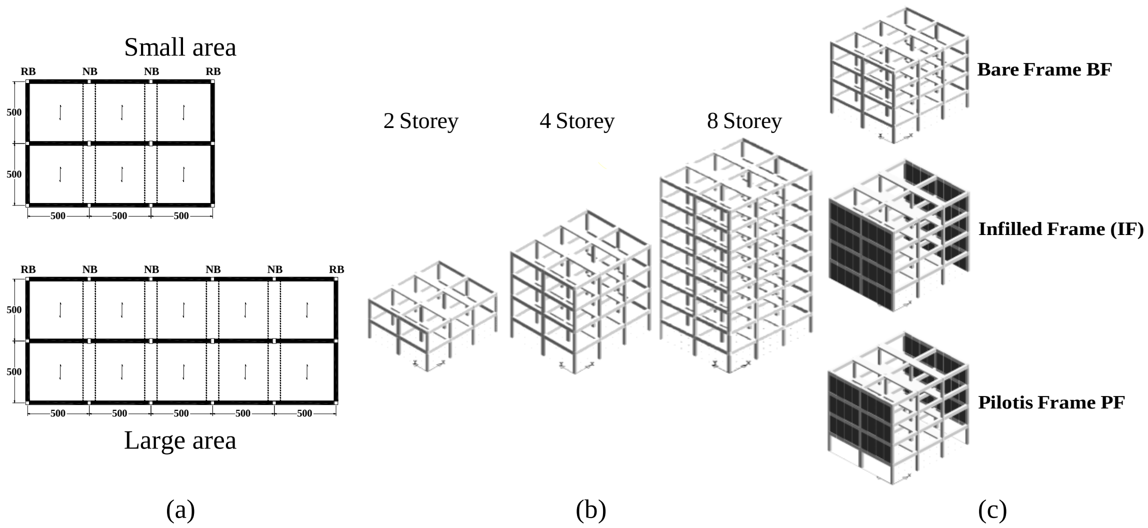


Fig. 1 Plan dimensions (a), number of storeys (b) and infill distributions (c) of the building types considered in the study

structure can be neglected), Infilled Frame types (IF, frames with effective and regularly arranged masonry infills), and Pilotis Frame types (PF, frames without masonry infills at the ground floor).

Cross-section dimensions and reinforcement details of the resisting members derive from simulated design taking into account only gravity loads (Masi 2003) Reference is made to the structural codes for RC buildings in effect in Italy before and after 1971, when a significant change took place in the Italian structural code. Thus, two construction periods have been considered, before 1971 (Ante71 in the following) and after 1971 (Post71). Note that the simulated design procedure adopted for Ante71 structures is not very different from the one adopted for Post71 ones, the main difference being in the mechanical properties of constituent materials.

Typical properties of materials relevant to the standards of each period of construction have been assumed, specifically low quality concrete (C10/12 with mean compressive strength f_{cm}=16 MPa) and smooth steel (with mean yielding strength f_{ym}=400 MPa) for Ante71 types, and medium quality concrete (C20/25 with f_{cm}=28 MPa) and deformed steel (close to S400 type with f_{ym}=400 MPa) for Post71 types. Internal force values have been computed on the basis of the characteristic values of dead and live loads. Live loads have been assumed as equal to 2.0 kN/m^2, as prescribed for residential buildings. In designing structural members, safety verifications have been performed according to the allowable stress method. The columns were designed by taking into account only axial load and adopting the minimum requirements regarding reinforcement provided in the typical codes of the period. The beams were designed on the basis of the simplified model of continuous beam resting on simple supports.

Masonry infills typically adopted in the Italian and European building stock have been considered (Braga *et al*. 2011). They are made of two layers of hollow brick masonry with poor mechanical characteristics, and a total gross thickness equal to about 30 cm for structures built both before and after 1971.

More details on the selection and characteristics of the structural types under study and on the simulated design carried out to detail them can be found in (Masi 2003, Masi and Vona 2012).

2.2 Modelling and numerical simulations

Building types have been modeled by adopting a lumped plasticity approach incorporated in the computer program IDARC 2D (Valles *et al*. 1996). At each end of all structural members a three-parameter hysteretic hinge based on the Park model (Park *et al*. 1987a, b), which is able to account for stiffness degradation, strength deterioration, and pinching effect, has been defined. The values of the degrading parameters for Ante71 and Post71 structures are reported in Tables 1 and 2, respectively. They were determined by referring to the work of Ghobarah *et al*. (1999), and to the experimental results obtained by Kunnath *et al*. (1995a, b), Liu and Park (2000), Pampanin *et al*. (2002), on sub-assemblages having details typically found in gravity load designed buildings. Specifically, the adverse effects of smooth bars typically used in Ante71 structures have been considered by modifying the model parameters proposed in Ghobarah *et al*. (1999) on the basis of the test results in Liu and Park (2000) and Pampanin *et al*. (2002).

As regards masonry infill modelling, each panel has been modeled by using a 2D finite element based on the Wen-Bouc model (Bouc 1967, Baber and Noori 1985). Panel dimensions in the models have been defined using the expression elaborated by Mainstone (1974), relevant to rectangular masonry panels inserted in RC frames.

Starting from the values of the concrete strength as provided by the standards generally adopted and the codes in force at the time of construction, as above described, different values of concrete

Table 1 Adopted values of degrading parameters for Ante71 RC buildings

	Stiffness degradation (α)	Strength deterioration (β)	Pinching effect (γ)
Beams (internal joints)	1.5	0.15	0.6
Beams (external joints)	1.5	0.15	0.7
Internal Columns	1	0.15	0.6
External Columns	1	0.15	0.4

Table 2 Adopted values of degrading parameters for Post71 RC buildings

	Stiffness degradation (α)	Strength deterioration (β)	Pinching effect (γ)
Beams	2	0.1	0.7
Columns	1.5	0.1	0.7

strength f_c have been assumed in evaluating the seismic capacity. Besides, these values are selected taking into account the real mechanical properties of the materials that can be actually found in existing structures, as described in (Masi *et al*. 2014) where a large database made up of about 1500 test results on concrete cores has been analysed. Specifically, three f_c values for each of the two different periods under examination have been adopted, that is f_c=7, 11, 13 MPa for Ante71 buildings, and f_c=10, 18, 28 MPa for Post71 ones. On the contrary, to account for the steel type mostly used in real existing buildings belonging to each period, only a steel strength value f_y has been considered for each period, that is f_y=250 MPa (smooth steel) for Ante71 and f_y=400 MPa (deformed steel) for Post71 buildings. In the derivation of FCs the above values have been assumed as deterministic and the aleatory variability has not been taken into account.

Two 2D-models, one in each direction of seismic motion corresponding to the principal axes of the structure (long. *X*, transv. *Y*), were prepared and analysed. Each 2D-model is made up of all the plane frames present in the related direction lined up and slaved at each floor (pseudo-3D models). This modelling is based on an equal displacement hypothesis at each floor, assuming that diaphragms exhibit sufficiently in plan stiffness to be modelled as rigid. As shown in Masi *et al*. (1997), such an assumption can be considered valid for RC floor slabs with dimensions and characteristics (e.g., absence of large openings or re-entrances) such as those present in the buildings under examination.

Structural response has been evaluated through Non-Linear Dynamic Analyses (NLDAs) making an appropriate selection of the seismic input to be applied in such a way as to effectively represent the damage potential of real seismic events (Masi *et al*. 2011, Chiauzzi *et al*. 2012). Specifically, 50 recorded accelerograms contained in the European Strong-Motion Database, ESMD (Ambraseys *et al*. 2004) and consistent with the damage potential of Italian-like earthquakes have been considered (Masi and Vona 2012). With regard to the earthquake intensity indicator, accelerograms were selected on the basis of an integral seismic parameter such as the Housner Intensity I_H, accounting for its higher capacity to represent the damage potential of ground motion. I_H is defined as follows

$$I_H = \int_{0.1}^{2.5} S_v(T, \xi = 0.05)\, dT \tag{1}$$

where $S_v(T, \xi = 0.05)$ is the spectral pseudo-velocity value evaluated for vibration period values T in the range 0.1-2.5sec, and ξ is the fraction of critical damping assumed equal to 5%. Details on

Table 3 Parameters of the Fragility Curves for Ante71 building types

		Ante71					
		BF		IF		PF	
Hight	***ds***	**median**	***β***	**median**	***β***	**median**	***β***
2 storey	1	0.31	0.25	0.40	0.32	0.23	0.30
	2	0.55	0.25	0.64	0.32	0.40	0.30
	3	0.82	0.25	0.95	0.32	0.53	0.30
	4	1.13	0.25	1.23	0.32	0.85	0.30
	5	1.57	0.25	2.14	0.32	1.37	0.30
	ds						
4 storey	1	0.25	0.28	0.40	0.22	0.25	0.29
	2	0.50	0.28	0.62	0.22	0.42	0.29
	3	0.78	0.28	0.82	0.22	0.74	0.29
	4	1.02	0.28	1.13	0.22	0.98	0.29
	5	1.40	0.28	1.69	0.22	1.23	0.29
	ds						
8 storey	1	0.38	0.33	0.40	0.21	0.36	0.23
	2	0.53	0.33	0.55	0.21	0.50	0.23
	3	0.93	0.33	0.95	0.21	0.85	0.23
	4	1.10	0.33	1.12	0.21	0.98	0.23
	5	1.47	0.33	1.57	0.21	1.41	0.23

Table 4 Parameters of the Fragility Curves for Post71 building types

		Post71					
		BF		IF		PF	
Hight	***ds***	**median**	***β***	**median**	***β***	**median**	***β***
2 storey	1	0.35	0.25	0.41	0.36	0.20	0.35
	2	0.55	0.25	0.66	0.36	0.40	0.35
	3	0.93	0.25	1.00	0.36	0.57	0.35
	4	1.23	0.25	1.33	0.36	0.96	0.35
	5	2.02	0.25	2.34	0.36	1.69	0.35
	ds						
4 storey	1	0.28	0.28	0.40	0.23	0.25	0.29
	2	0.50	0.28	0.62	0.23	0.42	0.29
	3	0.78	0.28	0.82	0.23	0.74	0.29
	4	1.12	0.28	1.21	0.23	1.03	0.29
	5	1.63	0.28	1.80	0.23	1.57	0.29
	ds						
8 storey	1	0.38	0.28	0.40	0.23	0.36	0.28
	2	0.53	0.28	0.55	0.23	0.50	0.28
	3	0.95	0.28	1.00	0.23	0.85	0.28
	4	1.12	0.28	1.21	0.23	1.10	0.28
	5	1.69	0.28	1.79	0.23	1.57	0.28

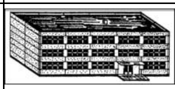
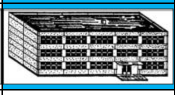
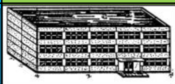
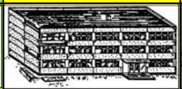
ds	0	1	2	3	4	5
EMS-98 definition						
	SD=null NSD=null	SD=null NSD=slight	SD=slight NSD=moderate	SD=moderate NSD=heavy	SD=heavy NSD=very heavy	Destruction
drift (%)	<0.1	0.1-0.25	0.25-0.5	0.5-1.0	1.0-1.5 (Ante71) 1.0-2.5 (Post71)	>1.5 (Ante71) >2.5 (Post71)

Fig. 2 Assignation of damage states *ds* (EMS 98) on the basis of drift values (SD=Structural Damage, NSD=Non Structural Damage)

the main parameter values of the selected accelerograms are reported in Table A1.

2.3 Damage scale

On the basis of the seismic response of the considered types, damage has been classified according to the criteria given in the European Macroseismic Scale EMS98 (Grünthal 1998), whose damage grades *ds* range from no damage (*ds*=0) to complete destruction (*ds*=5). Damage grades have been assigned as a function of interstorey drift, considering both structural and non structural components (Fig. 2).

The relationship assumed between the damage variable and the global damage states was originally proposed in (Masi 2003) for Post71 buildings, mainly derived from experimental and numerical results reported in the literature (Park and Ang 1985, Kunnath *et al*. 1995a, b, FEMA-NIBS 1999, Ghobarah *et al*. 1999). In order to take into account different design and construction characteristics of the RC structures under study as well as the results found during some recent experimental campaigns (e.g., Masi *et al*. 2013), the relationship adopted in (Masi 2003) has been updated assuming different drift values for *ds*4 and *ds*5 with respect to Ante71 and Post71 building types. Specifically, for Ante71 buildings, *ds*4 is assigned for drift values in the range 1.0-1.5%, thus *ds*5 is reached for values larger than 1.5%. Considering Post71 buildings, *ds*4 is assigned for drift values in the range1.0-2.5%, thus complete destruction occurs for drift values higher than Ante71 buildings, that is over 2.5%. Besides, with respect to the role of masonry infills in defining the damage levels, threshold values have been assumed on the basis of some experimental studies on infilled RC frames (Colangelo 2005, Calvi and Bolognini 2001, Hak *et al*. 2012), as proposed in (Manfredi and Masi 2014).

3. Generation of fragility curves

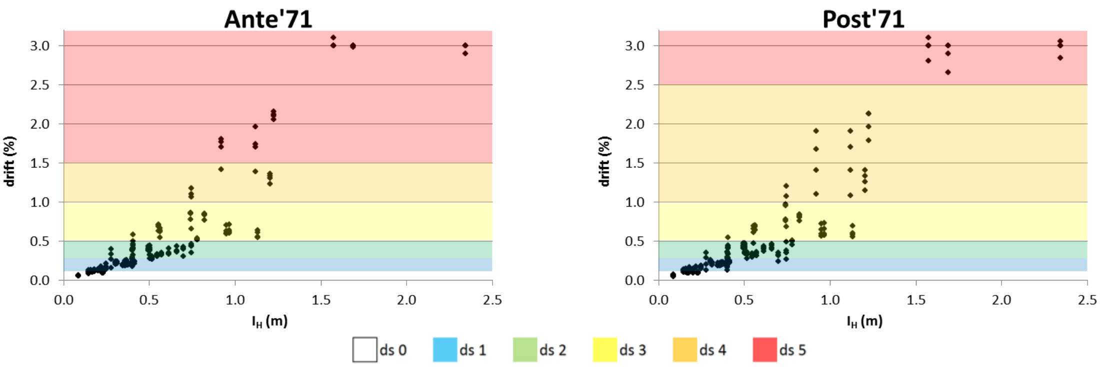
Fragility Curves have been derived from a wide parametric study, whose parameters (described at section 2) are summarized in the following:

- Period of construction (Ante71 and Post71);
- Storey numbers (2, 4, 8 storeys);
- Infill distribution (BF, IF, PF);

- Plan dimensions (small and large area);
- External Beams' Stiffness (RB, FB);
- Concrete strength (3 values for each considered period of construction).

As a result, a total of 216 building classes have been defined, each one subjected to 50 accelerograms applied separately along the two main horizontal directions in plan, thus performing a total of 21,600 NLDAs.

As discussed in Masi and Vona (2012), only some of the above parameters significantly affect the non-linear seismic behaviour of the building types under study. Specifically, the higher values of drift were always achieved along the weaker transversal direction *Y* (i.e., the shorter one) where frames are present on the external sides only. With respect to the role of the parameters adopted to classify structural types, results show that the variability due to plan dimensions and beam stiffness is rather low compared to other parameters. As a consequence, only the parameters referring to period of construction (2 cases), infill distribution (3 cases) and storey number (3 cases) have been considered as crucial, thus generating FCs relevant to 18 different building types (9 FCs for each construction period). Finally, a specific discussion is required for concrete strength. In order to take into account the effect of its variability in as-built real structures while acknowledging the practical impossibility of estimating concrete strength in large scale vulnerability studies, each FC has been defined by averaging the results deriving from the analyses carried out for the three strength values.



ds	Damage state	Drift/h (%)	N. of points
0	null	< 0.1	30
1	null	0.1 - 0.25	194
2	slight	0.25 - 0.5	177
3	moderate	0.5 - 1.0	104
4	heavy	1.0 - 1.5	23
5	Destruction	>1.5	72
	Total		**600**

ds	Damage state	Drift/h (%)	N. of points
0	null	< 0.1	87
1	null	0.1 - 0.25	182
2	slight	0.25 - 0.5	177
3	moderate	0.5 - 1.0	90
4	heavy	1.0 - 2.5	40
5	Destruction	>2.5	24
	Total		**600**

Fig. 3 Above: typical relationship IH - drift for BF type with 4 storeys belonging to Ante71 (on the left) and Post71 (on the right) class. Below: tables reporting the number of points (i.e., results from NLDA) considered for each damage states *ds*

3.1 Fragility curves

Fragility Curves represent the conditional probability of being in, $P[D=ds]$, or exceeding, $P[D>ds]$, a certain damage state *ds*. As stated above, five damage states, beyond the null state, have been considered based on the damage classification of the EMS98 (Grünthal 1998).

In line with the methodology provided in the RISK-UE project (Milutinovic and Trendafiloski 2003), each FC is characterized by the median value and the lognormal standard deviation of the selected earthquake intensity indicator, i.e., the Housner Intensity I_H in the present study, according to the following expression

$$P[ds/IH] = \Phi\left[\frac{1}{\beta_{ds}} ln\left(\frac{IH}{IH_{med,ds}}\right)\right] \tag{2}$$

where:

$I_{H,med,ds}$ is the median value of I_H at which the structure reaches a certain threshold of the damage state *ds*;

β_{ds} is the standard deviation of the natural logarithm of I_H at a certain damage state *ds*;

Φ is the standard normal cumulative distribution function.

For each NLDA carried out on the building types under study, the maximum drift value evaluated at a certain storey, suitably selected to represent global damage state, has been referred to the I_H value of the corresponding accelerogram. In this way, a I_H-drift relationship has been defined for each building type subjected to all the considered accelerograms. An example is displayed in Fig. 3 for a BF type with 4 storeys where also the number of points considered in the above described statistical analyses is reported.

For each relationship, by considering the threshold drift values defining each damage state, the points I_H-drift falling into the different drift ranges have been identified. Consequently, the median value $I_{H,med,ds}$ and the standard deviation β_{ds} of the I_H values belonging to the different ranges (damage states) have been determined.

As expected, the results of the analyses provide median values $I_{H,med,ds}$ which increase with the severity of the damage state (from *ds*0 to *ds*5), while an incoherent trend has been found for the standard deviation values β_{ds}. Different values of β_{ds} associated with each damage state produce unrealistic shapes of FCs thus computed (for instance, curves relevant to different damage states intersect themselves). Therefore, as already carried out for other studies (e.g., Kappos *et al*. 2006), a constant β_{ds} value has been assumed for all damage states in each building type, calculating it as the mean value of the β_{ds} values associated with each damage state. $I_{H,med,ds}$ and β_{ds} values defining the various FCs are summarized in Tables. 3 and 4 for Ante71 and Post71 building types, respectively. The related FCs are displayed in Figs. 4 and 5.

As expected, Post71 types generally show better performances than Ante71 types. As an example, the probability that 50% of 2storey-BF buildings reach or exceed *ds*3 requires I_H values equal to 0.82 and 0.93 for Ante71 and Post71 type, respectively. This I_H value goes up to 1.57 (Ante71) and 2.02 (Post71) to reach *ds*5. Differences decrease in both taller and regularly infilled types.

Considering a given I_H value, for both Ante71 and Post71 building classes, PF types generally show levels of expected damage higher than both IF and BF types. Specifically, more remarkable differences between PF and both IF and BF types can be found for 2-storey types with respect to

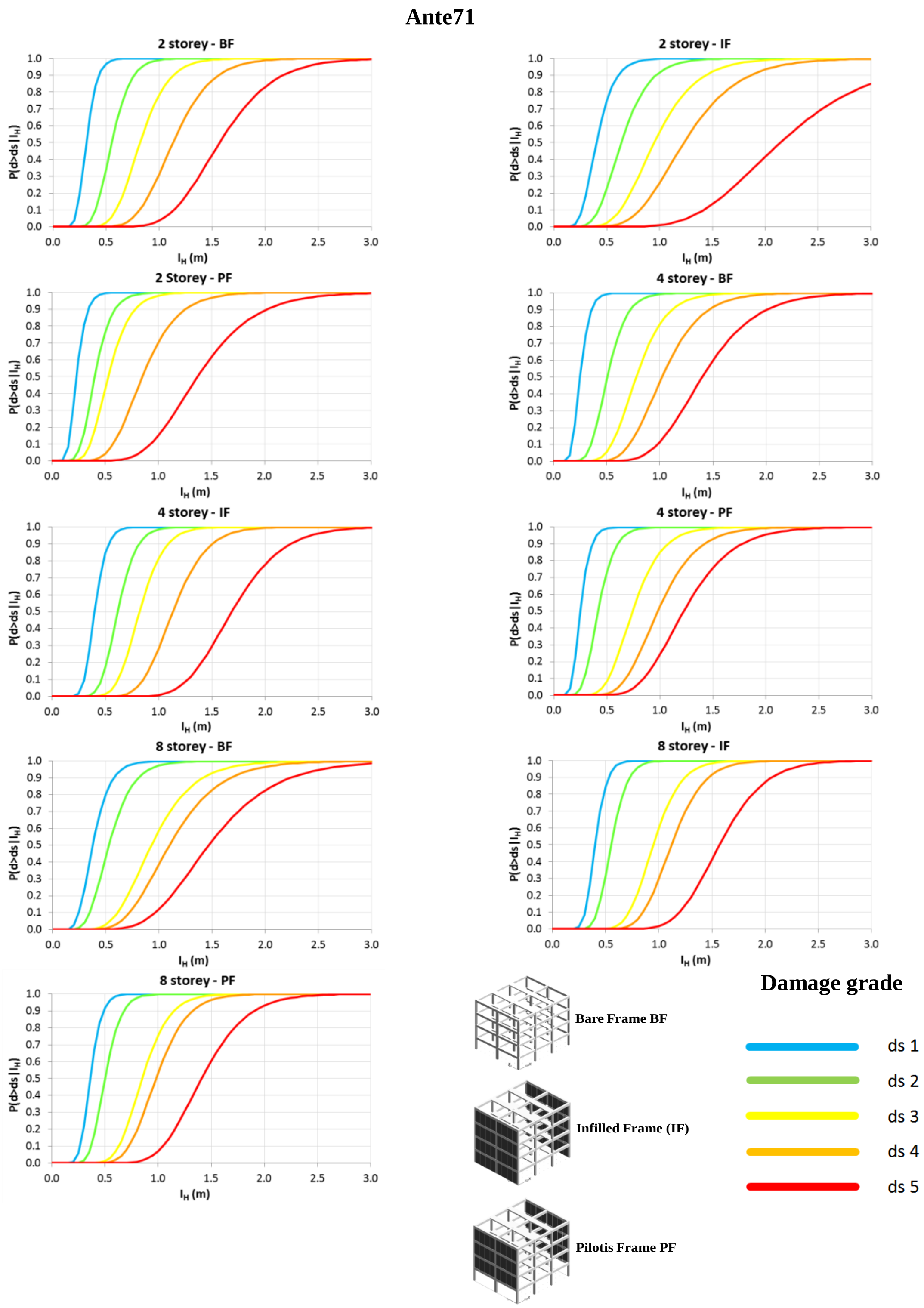


Fig. 4 Fragility Curves for Ante71 building types

Post71

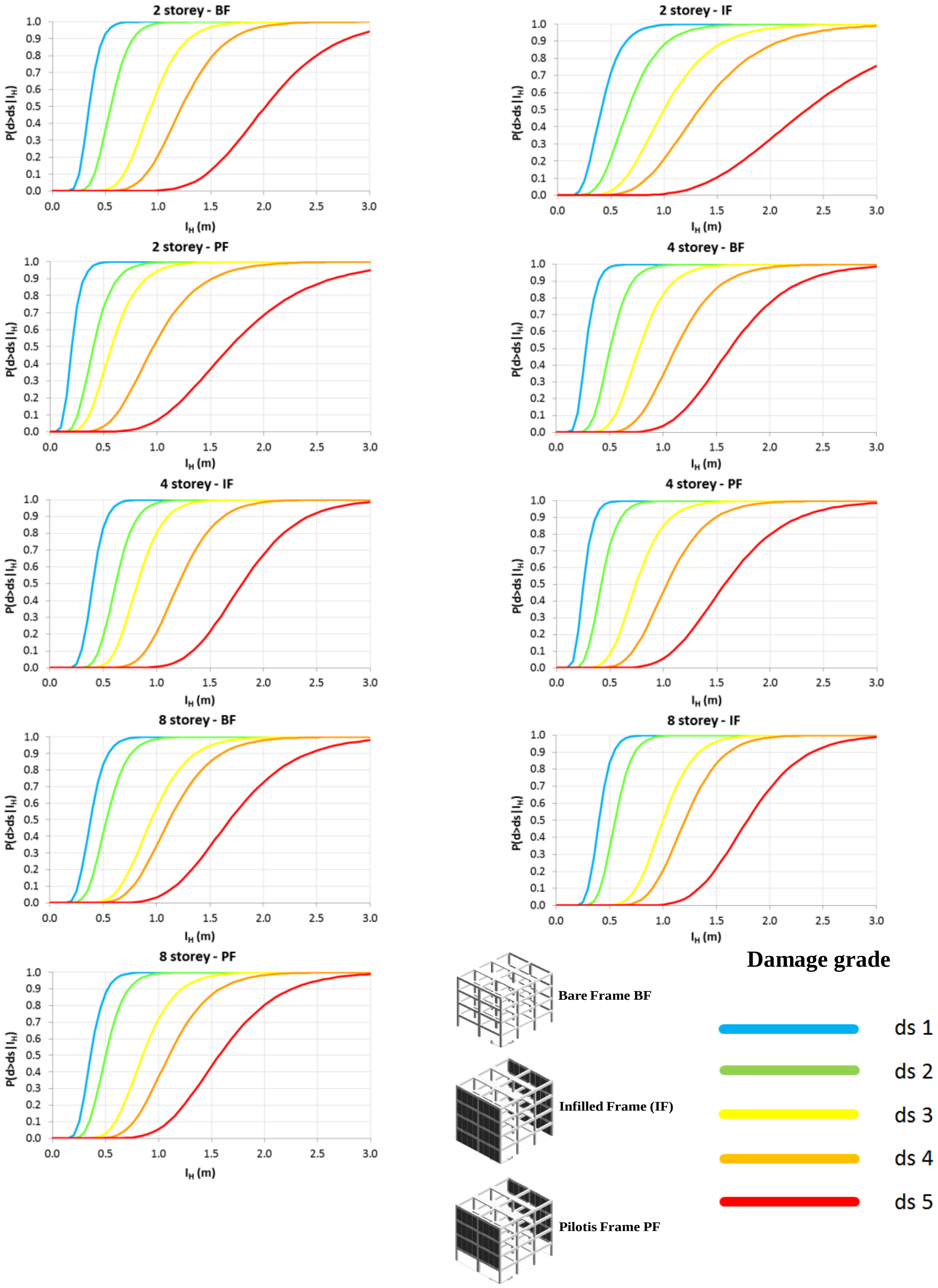


Fig. 5 Fragility Curves for Post71 building types

4- and 8-storey types. As an example, the probability that 50% of 2storey-Ante71 buildings reach or exceed *ds*3 requires I_H values equal to 0.57 and 1.00 for PF and IF type, respectively. This IH value goes up to 1.37 (PF) and 2.14 (IF) to reach *ds*5.

Among the considered structural parameters, infill distribution shows the greatest influence on seismic response thus confirming the remarkable contribution generally provided by regularly arranged infills in reducing seismic vulnerability of gravity-load designed buildings.

4. Comparison with other fragility studies and damage observed in past earthquakes

In order to verify the present study with respect to other fragility studies given in the technical literature, a comparison was made between the proposed FCs and those deriving from:

(i) the UNIGE (Università degli Studi di Genova (UNIGE), Italy) approach developed within the RISK-UE project (Milutinovic and Trendafiloski 2003) and updated in (Lagomarsino and Giovinazzi 2006), and

(ii) the AUTH (Aristotle University of Thessaloniki, Greece) approach (Kappos *et al.* 2006) given in the PAGER report (D'Ayala *et al*. 2012).

Note that these approaches were purposely selected because they provide results on RC frames without seismic design (Pre-code buildings).

The comparison was made on the basis of the expected damage distributions and mean damage index (DI_{med}) on similar building types, as defined in each different approach. As reported in (Dolce *et al.* 2006), mean damage index DI_{med} is defined as

$$DI_{med} = \frac{\sum_i (ds_i \cdot f_i)}{n} \tag{3}$$

where ds_i is a generic damage grade (ds_i=1-5), f_i is the related frequency. The summation is carried out for n=5, i.e., zero damage grade is not included. In this way, DI_{med} varies between 0 and 1, where DI_{med}=0 means total absence of damage and DI_{med}=1 means total destruction.

Expected damage was evaluated with respect to three different intensity levels (i.e., low, moderate, high). In order to take into account the effects due to the record-to-record variability, for each intensity level 10 accelerograms with I_H values around 0.25 (low), 0.50 (moderate) and 1.0 (high) were selected among the records listed in Table A1. An average damage distribution for each intensity level was evaluated by averaging the results obtained from 10 accelerograms.

Specifically, for the proposed approach (named USB-IT in the following), the expected damage distributions were computed in correspondence with the I_H values computed from each selected accelerograms, by adopting the FC relevant to the appropriate building type (Fig. 6).

For both RISK-UE and PAGER approach, seismic intensity is defined in terms of spectral displacements and the FCs are generated on the basis of the median value of the spectral displacement corresponding to the threshold of each damage state and of the variability associated with that damage state. As a consequence, in accordance with the Capacity Spectrum Method (Freeman 1998), for each accelerogram the spectral displacement corresponding to the "performance point" was evaluated at the intersection point between the capacity curve (relevant to each building type) and the demand spectrum, each one represented in ADRS (acceleration-Displacement Response Spectra) coordinate system. On the basis of the spectral displacement above described, the damage distributions have been determined using the pertinent FC.

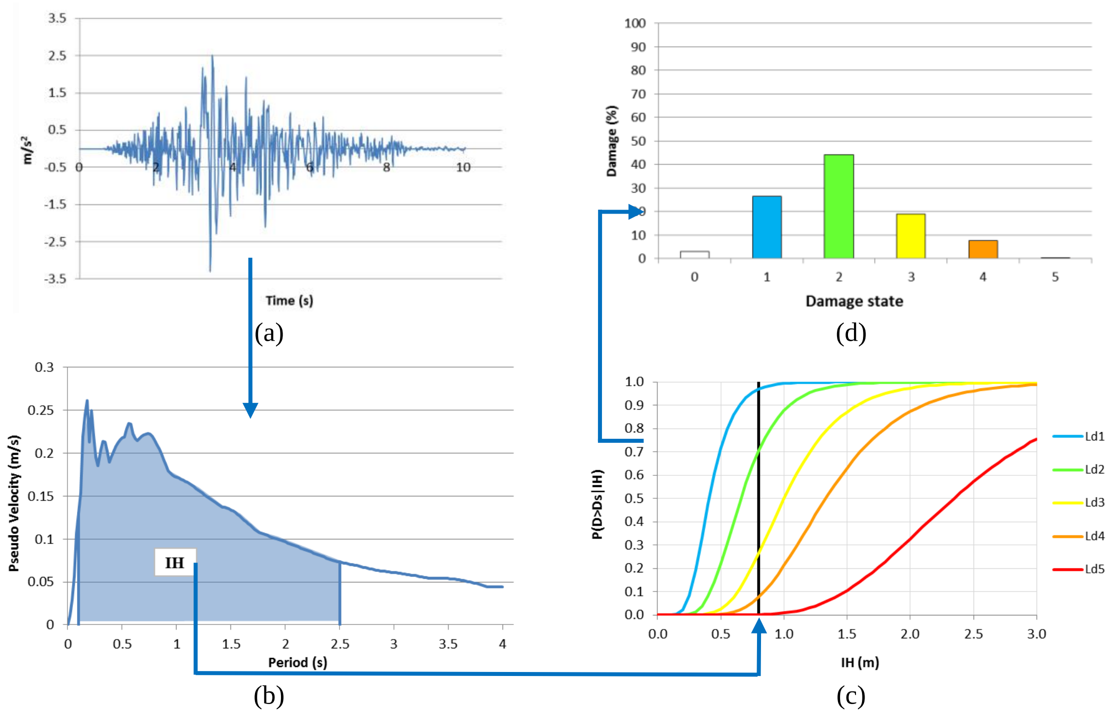


Fig. 6 Outline of the steps to achieve the damage distribution: (a) accelerogram, (b) pseudovelocity spectrum, (c) Fragility Curves, (d) damage distribution

It is worth noting that the FCs developed in the RISK-UE approach are based on a different damage scale with respect to the USB-IT and PAGER approach. In particular, the RISK-UE approach refers to four damage grades. As a consequence, when comparing results from the RISK-UE approach, *ds*4 and *ds*5 damage grades from the USB-IT approach have been merged and Eq. (3) to compute DI_{med} has been consequently modified by considering n=4. No modifications have been required in the PAGER *vs* USB-IT comparison.

Finally, FCs from the USB-IT and AUTH-PAGER studies have been further analysed by comparing their results with the damage distributions observed after the devastating Southern Italy 1980 earthquake, where data on about 3,000 RC buildings were collected and analysed (Braga *et al*. 1982, Masi *et al*. 2000). More recent Italian earthquakes (e.g., Molise 2002, L'Aquila 2009) have not been considered because of the scarcity of affected RC buildings (Mucciarelli *et al*. 2003) or the need to more thoroughly analyse the observed damage distribution.

4.1 Comparison with the UNIGE approach in RISK-UE

Only some building types were studied in the UNIGE approach, that is RC frames without infill walls known as RC1 type. This has made possible to compare expected damage on 2-, 4-, 8-storey types with BF configuration belonging to Ante71 class (USB-IT approach) with that from the analogous types with varying height belonging to Pre-Code class which includes buildings designed without seismic criteria (UNIGE approach, Milutinovic *et al.* 2003). In fact, as described in the WP1 report of the RISK-UE project (Lungu *et al.* 2001), the classification adopted in the

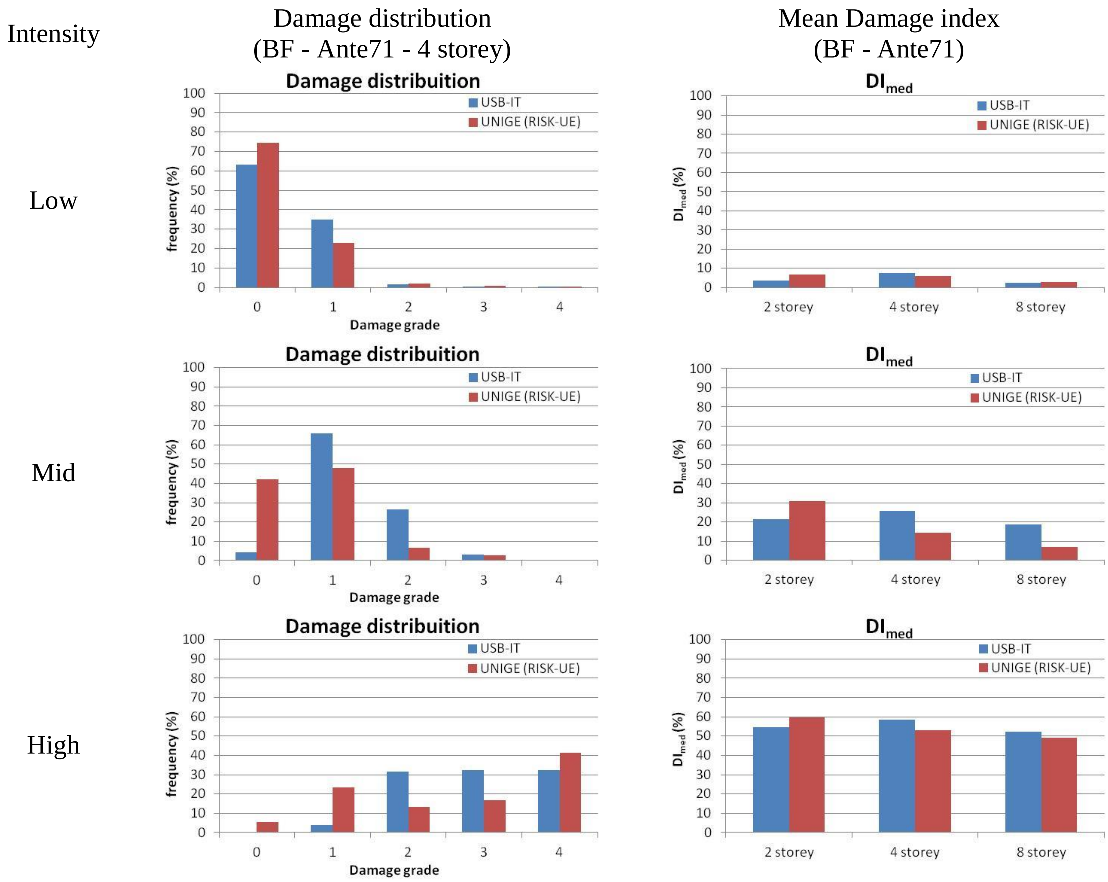


Fig. 7 Comparison between the USB-IT and RISK-UE approach in terms of damage distribution for BF-Ante71-4storey and DI_{med} for BF-Ante71 considering low-, mid- and high-intensity ground motion

present study essentially corresponds to that of UNIGE approach, that is 2, 4 and 8 storey types can be considered representative of low-, mid- and high-rise buildings, known as RC1L, RC1M and RC1H in the UNIGE approach, respectively. In the UNIGE approach the seismic response is evaluated through non linear static analyses by adopting the Capacity Spectrum Method (Freeman 1998). The spectral displacements related to the "performance point" identified for the types under study from each considered accelerogram are reported in Table A2.

For the sake of brevity, damage distributions have been compared only for the 4-storey type (Fig. 7, on the left), while comparisons for all building heights have been made in terms of mean index damage, DI_{med} (Fig. 7, on the right).

In terms of damage distribution, diagrams in Fig. 7 show significant differences when comparing the results obtained from UNIGE and USB-IT approach. Specifically, for mid-intensity ground motion, damage prediction from the USB-IT approach mainly involves the grades *ds*1-*ds*2 whereas, for the UNIGE approach, a greater frequency of the damage grades *ds*0 and ds1 is expected. For high-intensity, according to the proposed FCs, the damage grades from *ds*2 to *ds*4 are prevailingly expected with a similar frequency value (about 32%), whereas, for the UNIGE approach, the higher frequency values can be expected for the grade *ds*1 (23%) and *ds*4 (41%).

Lower differences can be found under low-intensity ground motion.

With respect to the results in terms of DI_{med}, lower differences can be found, particularly for high-intensity ground motion. Higher differences arise for mid-intensity where, for 4- and 8-storey type, the UNIGE approach provides lower values of DI_{med} with respect to those obtained by the USB-IT approach.

Summarizing, the two approaches show significant differences with respect to damage distributions. On the contrary, slight differences are found in terms of mean damage index, with the proposed FCs predicting lower damage grades for 2-storey type, while the contrary happens for 4- and 8-storey type.

4.2 Comparison with the AUTH approach in PAGER

Expected damage on BF, IF, PF types belonging to Ante71 building class with 4 storey (for the USB-IT approach) was compared with that on the corresponding types C4M, C3M, C3M-SS designed only to vertical loads for the AUTH approach (Kappos *et al.* 2006) given in the PAGER report (D'Ayala *et al*. 2012). Types considered in the PAGER report represent Italian RC buildings with 4 storey and different infill configuration in elevation, that is types without infill walls (C4M vs BF type), with infill walls at all storeys (C3M vs IF type) and without infill walls at the first storey (C3M-SS vs PF type).

As for the UNIGE approach, in the AUTH approach the seismic response is evaluated through non linear static analyses by adopting the capacity spectrum method (Freeman 1998). The spectral displacements related to the "performance point" identified for the types under study from each considered accelerogram are reported in Table A2.

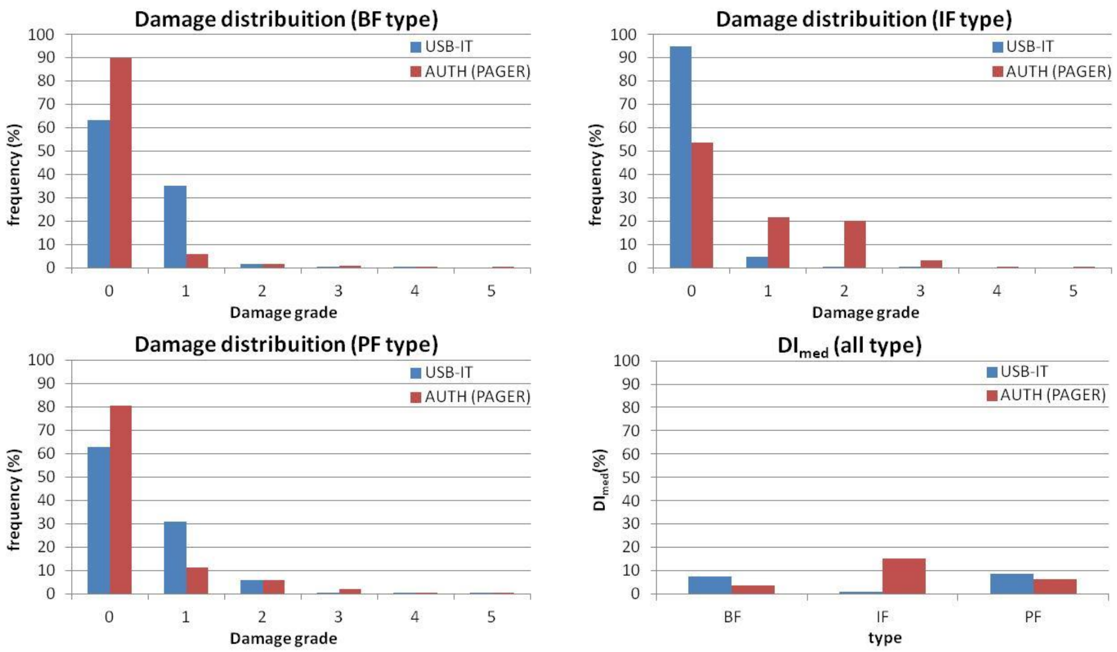


Fig. 8 Comparison between the USB-IT and AUTH (PAGER) approach in terms of damage distribution and mean damage index (DImed) for BF, IF, PF 4-storey Ante71 types considering low-intensity ground motion

Comparisons between damage distributions obtained from the USB-IT and the AUTH approach are displayed in Figs. 8, 9 and 10 for low-, mid- and high-intensity, respectively.

A consideration of ground motion with the lower intensity (Fig. 8) shows that the damage estimated in BF and PF types through the AUTH approach is lower than that obtained from the USB-IT approach. Opposite results are found for regularly infilled frames (IF type). Specifically, for IF type, it is worth noting that the proposed FCs predict negligible damage, while the use of the AUTH approach shows that higher damage states should be expected, up to partial or total collapse (*ds*4-*ds*5), although only in a very few cases. Consequently, with regard to the results in terms of DI_{med}, the value obtained from the AUTH approach is lower for BF type (0.04 vs 0.08), and PF type (0.07 vs 0.09), while DI_{med} values remarkably higher than those from the proposed approach are found for IF type, that is the USB-IT FCs provide a negligible value of DI_{med}, whereas a value of around 0.15 can be computed from the AUTH approach.

A consideration of the mid-intensity ground motion (Fig. 9) shows that the expected damage for IF type obtained from the proposed FCs mainly involves the lower damage states (*ds*1 and *ds*2), while heavier damage states should be expected when utilizing the AUTH approach. Opposite results are found for BF and PF types: the USB-IT approach provides higher values of *ds*2-*ds*3 damage states whereas, for the AUTH approach, null or negligible damage grades are mainly expected. As a result, higher DI_{med} values are computed for BF and PF types through the USB-IT approach, that is 0.26 vs 0.08 (BF type) and 0.30 vs 0.20 (PF type), while the contrary happens for IF type (0.14 vs 0.38). Note that DI_{med} provided by the AUTH approach for IF type is far higher than those for both BF and PF types, thus showing poor capability in taking into account the favorable contribution of regularly arranged infills on the seismic behavior of RC framed buildings.

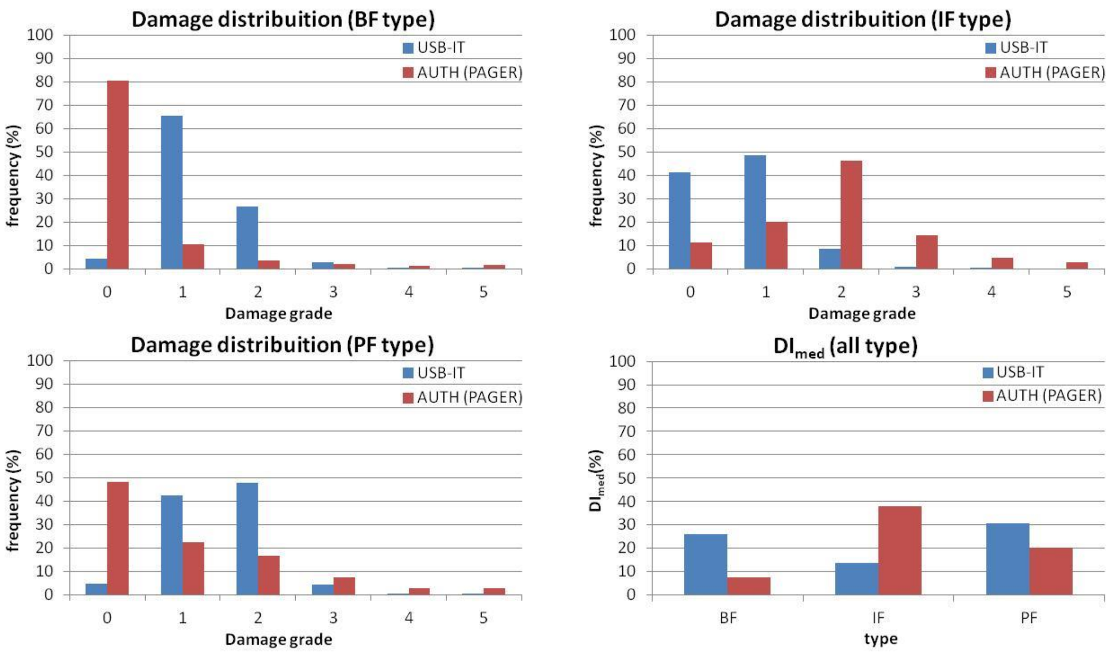


Fig. 9 Comparison between the USB-IT and AUTH (PAGER) approach in terms of damage distribution and mean damage index (DImed) for BF, IF, PF 4-storey Ante71 types considering mid-intensity ground motion

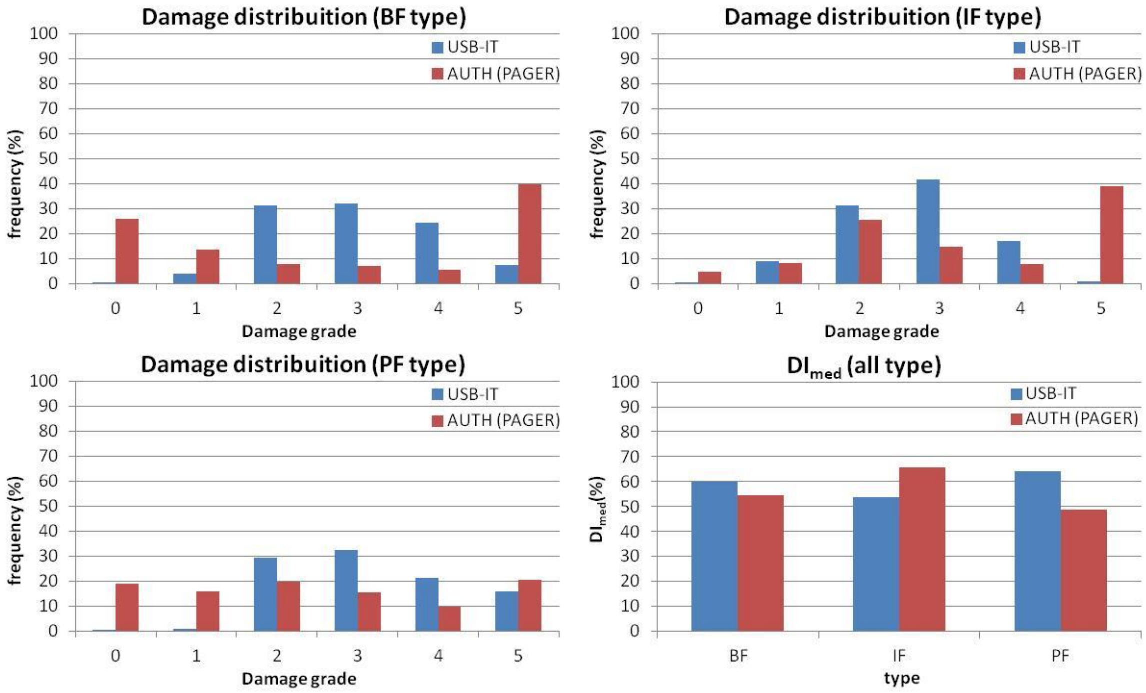


Fig. 10 Comparison between the USB-IT and AUTH (PAGER) approach in terms of damage distribution and mean damage index (DI_{med}) for BF, IF, PF 4-storey Ante71 types considering high-intensity ground motion

Finally, for the high-intensity ground motion (Fig. 10), by applying the AUTH approach remarkably higher frequency values for *ds*5 are found, thus presumably overestimating expected damage, particularly for IF types. On the contrary, the FCs of the USB-IT approach provide higher values for the other damage states (*ds*2-*ds*4). With respect to DI_{med}, close results between the two considered approaches are found, especially for BF type.

Summarizing, the proposed FCs generally provide higher damage grades with respect to AUTH-based results for bare and pilotis frames. FCs from AUTH appears to assign very high vulnerability to regularly infilled buildings, resulting in unexpected heavy damage predictions even for low-intensity ground motions.

4.3 Estimated vs observed damage

Comparisons made between the proposed FCs and other fragility studies have shown remarkable differences when considering different seismic intensities and different building types (e.g., either bare or infilled frames). To better understand and discuss such differences a comparison with the damage observed in past earthquakes on building types similar to those considered in the above approaches has been carried out.

To this end, observed damage on RC buildings after the 1980 Southern Italy earthquake provides an interesting benchmark. After this earthquake about 38,000 buildings were surveyed involving all the dwelling buildings of 41 municipalities present in the affected area. As a result, the Damage Probability Matrices (DPM's) of the most common building types were evaluated from the obtained database (Braga *et al*. 1982). Within this database, about 3,000 buildings had RC structure thus providing a prominent set of damage data. The main characteristics and performances of these buildings were reported and analysed in Masi *et al.* (2000). Specifically, damage distributions for three different seismic intensities, that is VI, VII and VIII EMS macroseismic intensity, were provided by considering all the RC buildings in the affected area.

In view of the comparison between estimated and surveyed damage, some preliminary remarks need to be made:

• the proposed FCs are provided separately for BF, IF and PF types, thus their relative presence in the 1980 DB needs to be defined; data from surveys carried out in the affected area (e.g., Stratta *et al*. 1981) suggests the following distribution: 15% BF, 70% IF and 15% PF type;

• in the analytical FCs seismic intensity is defined in terms of I_H or spectral displacement values, whereas, in the 1980 DB, EMS seismic intensities are considered, therefore an equivalence is required to make possible the comparison; by considering the results in Chiauzzi *et al*. (2012), the low-, mid- and high-intensity ground motions considered in the present study can be considered averagely equivalent to VI, VII and VIII EMS intensity, respectively;

• most of the 3,000 RC buildings reported in the 1980 DB were in places where VI or VII EMS intensity was assigned (about 90%); whereas for VIII intensity, only two towns were assigned such intensity (i.e., Lioni and S. Angelo dei Lombardi), one of them was already seismically classified before 1980 thus making the comparison with the proposed FCs relevant to pre-code buildings inappropriate.

On the basis of the above premises, in the following a comparison between 1980 observed damage and damage estimated by USB-IT and AUTH-PAGER approach is made considering low- and mid-intensity.

Results are shown in Fig. 11 in terms of damage distribution and mean damage index. In general, remarkable differences appear, with damage grades from the considered FCs generally higher than those observed in the 1980 earthquake.

Specifically, considering the lower intensity (VI EMS), damage estimated through the AUTH approach is higher than that obtained from the 1980 survey, the former providing non negligible probability of damage up to *ds*3.

By applying the proposed FCs (USB-IT) lower damage is averagely estimated with a very large probability of null damage. Consequently, with regards to the results in terms of DI_{med}, the value obtained from the AUTH approach is higher compared to surveyed damage (0.12 vs 0.09), while the contrary happens with the USB-IT approach (0.03 vs 0.09).

Considering the mid-intensity ground motion (VII EMS), expected damage from both USB-IT and AUTH approach overestimates surveyed damage, with higher differences from the latter approach. USB-IT approach overestimates expected frequency of *ds*1 and *ds*2, while giving acceptable results for *ds*3-*ds*5. As a result, higher DI_{med} values are computed for both fragility models, although with a lower difference from the USB-IT approach (0.18 vs 0.13) with respect to the AUTH approach (0.30 vs 0.13).

The comparison confirms the difficulty, already discussed in other studies (e.g., Colombi *et al*. 2008, Erberik 2008), in adequately predicting real damage distributions through fragility models, even when based on accurate models as in the present study. In general, the examined fragility models provided damage distributions of greater severity than those found in real earthquakes, thus confirming the tendency of the analytically-derived fragility or vulnerability models to be more conservative than the observed damage data (Colombi *et al*. 2008).

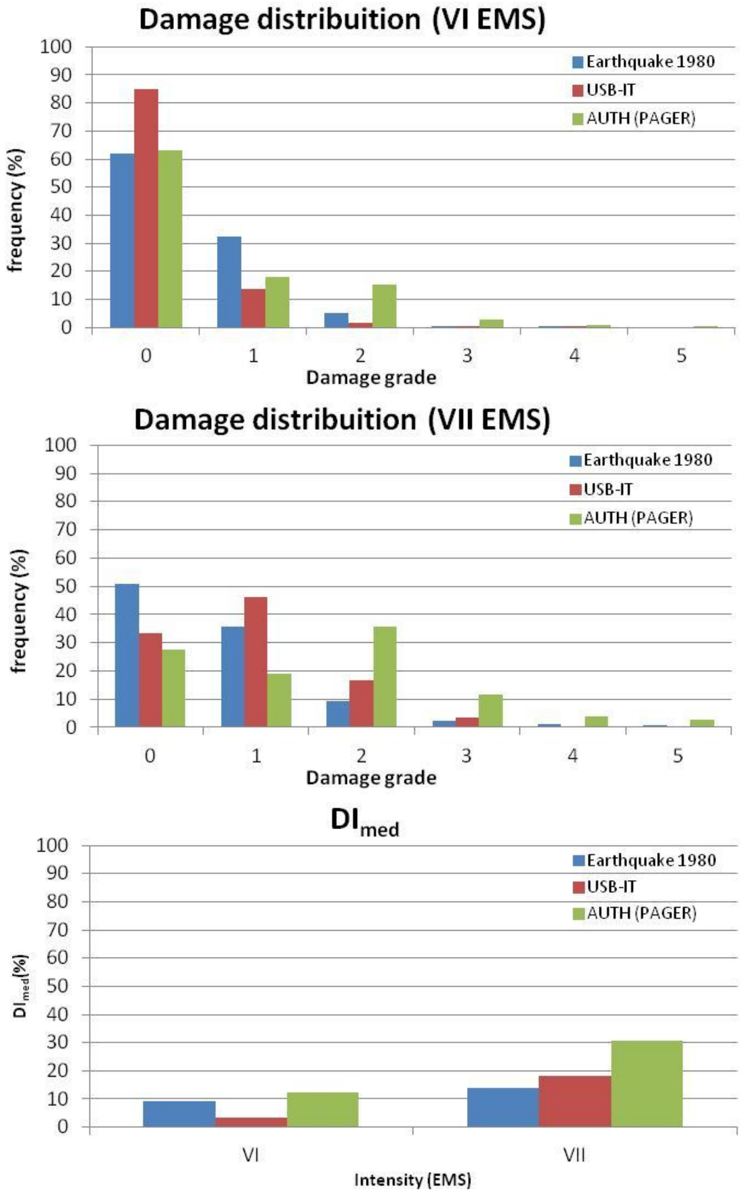


Fig. 11 Comparison between surveyed damage data (Southern Italy earthquake 1980) and estimated damage data (USB-IT and AUTH-PAGER approach) in terms of damage distribution and mean damage index (DI_{med})

However, lower differences have been found when dealing with the proposed FCs, thus encouraging further developments currently in progress with a view to improving them. To this end, emphasis should be placed on the major role of seismic input that can significantly influence the results of the comparison between estimated vs observed damage, particularly in relation to the large inherent scatter of the correlation between instrumental intensity measures, such as I_H and S_d in the fragility models, and macroseismic intensities (Masi 2003, Chiauzzi *et al*. 2011).

5. Discussion and final remarks

In order to support seismic risk mitigation policies and civil protection activities, effective tools able to estimate the real vulnerability of buildings in prone-earthquake areas are required. To this end, analytical methods such as fragility curves (FCs), which provide the probability that the seismic demand of building types under given seismic ground motions reaches or exceeds various states of damage, can be effectively used.

In this paper, a set of FCs relevant to regular RC building types representative of the Italian building population designed only to vertical loads has been derived from an extensive campaign of non-linear dynamic analyses. In particular, by varying different parameters such as number of storeys, presence and position of infill walls and period of construction (ante- and post 1971 year), nine sets of FCs according to the EMS98 damage scale have been defined in terms of Housner intensity.

Results firstly confirm the key role of infill walls on the vulnerability of buildings when they are irregularly arranged along the elevation (pilotis frames, PF). In fact, for the same ground motion intensity, PF types generally show levels of expected damage higher than both bare frames (BF) and regularly infilled frames (IF). Concerning the differences between the considered periods of construction, Ante71 types show higher damage grades than Post71 ones, even though differences decrease in the taller structures and in the IF types. Considering the number of storeys, more remarkable differences are found between 2- and 4-storey types, while for 8-storey types results appear to be very close.

Remarkable differences have been found by comparing the results obtained from the proposed FCs and those by some prominent studies in the available state-of-art, that is the RISK-UE and PAGER approaches. Expected damage grades obtained from the proposed FCs are comparable to those provided by the RISK-UE approach, at least in terms of mean values. On the contrary, large differences are found with the PAGER approach which provides unrealistically higher damage predictions, especially regarding regularly infilled building types. In this respect, it should be emphasized that some of the differences found in the comparisons could derive from a different treatment of uncertainties in the different approaches.

To better understand the remarkable differences found in the fragility studies, a comparison between their results and the damage observed in past earthquakes on similar RC building types has been performed. To this end, observed damage after the 1980 Southern Italy earthquake has been considered, where 3,000 RC frame buildings with various damage grades were surveyed. Such a comparison confirmed the difficulty in adequately predicting real damage distributions through fragility models which generally provide damage distributions of greater severity than those found in real earthquakes, although lower differences have been found when dealing with the FCs developed in the present study.

It is worth underlining that, even if the selected types derive from the Italian built environment, results can be extended to building types present in other parts of the world designed only to gravity loads and having similar material properties and levels of detailing. In fact, Ante71 types can be considered as representative of non ductile RC framed buildings with poor construction quality (e.g., with low concrete strength and smooth steel bars), whereas Post71 buildings can be considered as representative of non ductile RC framed buildings with medium construction quality (e.g., with medium concrete strength and deformed steel bars). Such building types are widely used also in other parts of the world. Specifically, RC framed structures with masonry infills are extensively used in many countries, e.g. they comprises approximately 75% of the building stock in Turkey, about 60% in Colombia, and over 30% in Greece (Yakut 2004).

Further studies are currently in progress to improve the proposed FCs. Specifically, as a consequence of the remarkable role of infill walls on seismic behaviour, future FCs should be developed taking into account the variability of dimensions and mechanical properties deriving from the different types of infill panels adopted in the building practice. Furthermore, in order to extend the set of FCs, buildings with in-plane irregularity deriving from either irregular plane shape or asymmetric distribution of resisting elements will be studied. Finally, in comparing estimated and observed data either more accurate correlations between instrumental and macroseismic intensity measures will be sought or, if possible, the benchmarking comparison will be based on damage data from areas where ground motion recordings are available (e.g., L'Aquila 2009).

Acknowledgements

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Appendix A

Table A1 List of records selected from the European Strong-Motion Database (Ambraseys *et al*. 2004).

ID accelerogram	**ID File**	**PGA [m/s^2]**	t_{max} **[s]**	I_H **[m]**
1	000172ya	0.37	24.6	0.09
2	000359ya	0.66	65.0	0.16
3	000980ya	0.78	40.5	0.14
4	000365xa	0.99	30.7	0.18
5	000316xa	1.32	33.6	0.17
6	000384xa	1.43	59.8	0.22
7	000361ya	1.57	28.3	0.17
8	000316ya	1.66	33.6	0.20
9	000363ya	1.85	26.1	0.30
10	000159xa	2.37	21.6	0.40
11	000134ya	2.14	22.0	0.40
12	000651ya	2.31	19.8	0.31
13	000159xa	2.37	21.6	0.40
14	000766xa	2.53	23.1	0.36
15	000766xa	2.61	23.1	0.37
16	000027ya	2.70	7.7	0.35
17	000770xa	2.75	56.2	0.23
18	000770xa	2.81	56.2	0.23
19	000027ya	2.94	7.7	0.38
20	000067ya	3.00	18.1	0.54
21	000067ya	3.15	18.1	0.57
22	000027ya	3.24	7.7	0.42
23	000766ya	3.29	23.1	0.49
24	000766ya	3.35	23.1	0.50
25	000501xa	3.40	36.3	0.95
26	000501xa	3.46	36.3	0.96
27	000027xa	3.74	7.8	0.70
28	000027xa	4.00	7.8	0.75
29	000593xa	4.30	28.7	0.62
30	000593xa	4.61	28.7	0.66
31	000126ya	4.96	10.0	0.78
32	000297ya	0.32	62.0	0.28
33	000295ya	0.55	50.0	0.40

Table A1 Continued

ID accelerogram	ID File	PGA [m/s^2]	t_{max} [s]	I_H [m]
34	000296xa	0.61	54.5	0.25
35	000049xa	0.61	42.7	0.41
36	000294xa	0.91	65.0	0.74
37	000612ya	0.93	65.0	0.92
38	000293xa	0.99	65.0	0.55
39	000289ya	1.35	65.0	0.52
40	000170ya	1.56	31.5	0.56
41	000600xa	1.59	57.0	0.50
42	000291ya	1.70	65.0	1.21
43	000287ya	1.80	65.0	1.23
44	000592xa	1.99	49.7	0.82
45	001231ya	2.24	54.0	1.12
46	001228xa	2.34	47.4	0.74
47	000290ya	3.00	61.0	1.57
48	001257xa	3.04	65.0	1.69
49	000055ya	3.09	35.9	1.13
50	001226ya	3.54	27.0	2.34

Table A2 Housner intensity and spectral displacement values used in the comparison between USB-IT, RISK-UE and PAGER approaches

		USB-IT	RISK-UE (BF types)			PAGER (4-storey types)		
		all types	RC1L	RC1M	RC1H	C4M	C3M	C3M-SS
Intensity	ID accelerogram	I_H (m)	S_d (cm)					
Low	2	0.16	0.52	0.74	1.02	0.95	0.52	0.74
	4	0.18	0.96	1.36	1.23	1.27	1.00	1.07
	5	0.17	0.48	0.66	0.91	0.91	0.50	0.43
	6	0.22	0.60	1.27	1.00	1.09	0.60	1.27
	7	0.17	0.34	0.45	0.67	0.69	0.38	0.39
	8	0.20	0.58	0.44	0.88	0.86	0.58	0.49
	9	0.30	0.75	2.13	2.02	1.96	0.75	1.48
	12	0.31	1.33	1.66	2.04	2.23	1.47	1.31
	16	0.35	1,96	1.55	1.17	1.24	1.90	1.80
	17	0.23	0.29	0.35	0.57	0.58	0.29	0.37
Mid	10	0.40	2.06	1.37	1.20	1.07	1.80	1.82
	11	0.40	1.29	1.26	2.90	2.35	1.23	1.09
	13	0.40	2.06	1.37	1.20	1.07	1.80	1.82
	14	0.36	1.33	1.56	1.55	1.55	1.37	1.44
	15	0.37	1.37	1.61	1.60	1.59	1.40	1.48
	19	0.38	2.14	1.69	1.27	1.35	2.02	1.96

Table A2 Continued

		USB-IT	RISK-UE (BF types)			PAGER (4-storey types)		
		all types	**RC1L**	**RC1M**	**RC1H**	**C4M**	**C3M**	**C3M-SS**
Intensity	**ID accelerogram**	I_H **(m)**	S_d **(cm)**					
Mid	20	0.54	2.67	2.22	1.65	1.72	2.54	2.65
	21	0.57	2.76	2.29	1.74	1.80	2.63	2.76
	22	0.42	2.32	1.86	1.40	1.49	2.19	2.16
	23	0.49	2.29	2.79	2.74	2.82	2.79	2.54
High	25	0.95	4.51	5.12	4.97	5.12	4.58	4.59
	26	0.96	4.51	5.20	5.05	5.20	4.66	4.66
	28	0.75	2.96	2.30	1.90	1.77	2.67	2.86
	31	0.78	2.30	3.54	2.93	3.04	4.22	4.16
	36	0.74	1.53	6.74	9.15	8.35	1.56	1.40
	37	0.92	1.05	2.00	9.15	8.73	1.20	2.18
	44	0.82	4.51	6.74	6.26	8.29	6.03	7.76
	45	1.12	4.51	6.74	9.15	8.73	6.03	6.27
	46	0.74	4.51	4.30	7.83	4.40	5.39	5.39
	49	1.13	4.51	5.75	4.48	5.75	6.03	7.26