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ORIGINAL RESEARCH PAPER

Deriving vulnerability curves using Italian earthquake damage data

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Abstract The concerted effort to collect earthquake damage data in Italy over the past 30 years has led to the development of an extensive database from which vulnerability predictions for the Italian building stock can be derived. A methodology to derive empirical vulnerability curves with the aforementioned data is presented herein and the resulting curves have been directly compared with mechanics-based vulnerability curves. However, it has been found that a valid comparison between the empirical and analytical vulnerability curves is not possible mainly due to a number of shortcomings in the database of surveyed buildings. A detailed discussion of the difficulties in deriving vulnerability curves from the current observed damage database is thus also presented.

Keywords Vulnerability curves · Damage data · Italian building stock · Loss estimation · Analytical methods

1 Introduction

The definition of the seismic vulnerability of buildings at an urban scale is a fundamental component of a loss model and much research has been carried out over the past 30 years in this field as summarised in Calvi et al. (2006). The first predictions of structural vulnerability at a large geographical scale were based on the observed damage from earthquakes in the form of damage probability matrices. Damage probability matrices express, in a discrete form, the conditional probability of obtaining a damage level j, due to a ground motion of

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intensity i, P[D $= j/i$]. Vulnerability functions, which are continuous functions expressing the probability of exceeding a given damage state given a function of the earthquake intensity, were later proposed, but were still based initially on the observed damage from earthquakes. There are many advantages of using observed data to study the vulnerability of the existing building stock, with the main advantage at the present time being the possibility to use such data to calibrate or attempt to validate analytically derived vulnerability curves.

The first damage probability matrices were developed by Whitman et al. (1973) and were based on the observed damage to different structural typologies from the 1971 San Fernando earthquake. One of the first European versions of a damage probability matrix was produced by Braga et al. (1982) and was based on the damage data of Italian buildings after the 1980 Irpinia earthquake. The buildings were separated into three vulnerability classes (A, B and C) and a DPM based on the MSK intensity scale was evaluated for each class. The use of DPMs is still popular in Italy and proposals have recently been made to update the original DPMs of Braga et al. (1982). Di Pasquale et al. (2005) have changed the DPMs from the MSK intensity scale to the MCS scale (Mercalli-Cancani-Sieberg) because the Italian seismic catalogue is mainly based on this intensity and the number of buildings has been replaced by the number of dwellings so that the matrices could be used in conjunction with the 1991 Census data, collected by ISTAT (Italian National Institute of Statistics). Dolce et al. (2003) have also adapted the original matrices by adding a vulnerability class D, using the EMS98 scale (Grünthal 1998), to account for the buildings that have been constructed since 1980, which should have a lower vulnerability as they would either have been retrofitted or designed to recent seismic design codes.

Continuous vulnerability functions based directly on the damage of buildings from past earthquakes were introduced slightly later than DPMs; one obstacle to their derivation being the fact that macroseismic intensity is not a continuous variable. This problem was overcome by Spence et al. (1992) through the use of their Parameterless Scale of Intensity (PSI) to derive vulnerability functions based on the observed damage of buildings using the MSK damage scale. Orsini (1999) also used the PSI ground-motion parameter to derive vulnerability curves for apartment units in Italy. Both studies subsequently converted the PSI to peak ground acceleration (PGA) using empirical correlation functions so that intensity was not being used both for the definition of the damage and the ground motion.

Sabetta et al. (1998) used post-earthquake surveys of approximately 50,000 buildings damaged by destructive Italian earthquakes in order to derive vulnerability curves. The database was sorted into three structural classes and six damage levels according to the MSK macroseismic scale. A mean damage index, calculated as the weighted average of the frequencies of each damage level, was derived for each municipality where damage occurred, and for each structural class. Empirical fragility curves with a binomial distribution were derived as a function of PGA, Arias Intensity and effective peak acceleration. Rota et al. (2006) have also used the post-earthquake damage surveys of approximately 90,000 buildings in Italy in order to derive typological fragility curves for typical building classes (e.g. seismically designed reinforced concrete buildings of 1–3 storeys). Observational damage probability matrices were first produced and then processed to obtain lognormal fragility curves relating the probability of reaching or exceeding a given damage state to the mean peak ground acceleration at the coordinate of the municipality where the damaged buildings were located. The PGA has been derived using the magnitude of the event and the distance to the site based on the Sabetta and Pugliese (1987) attenuation relation, assuming rock site conditions.

Alternative empirical vulnerability functions have also been proposed, generally with normal or lognormal distributions, which do not use macroseismic intensity or PGA to charac-

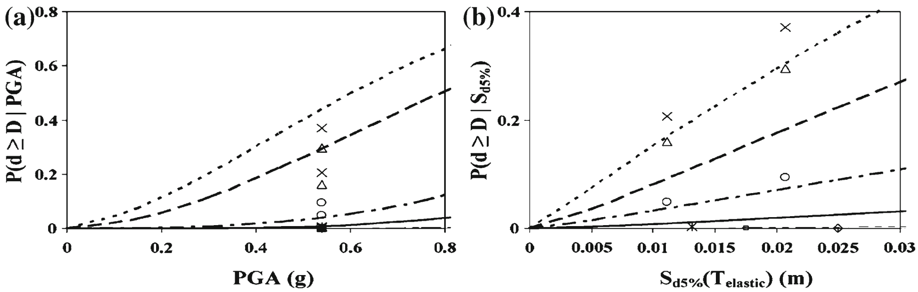

Fig. 1 Example of the difference in the vulnerability point distribution using observations of low and mid-rise building damage after the 1995 Aegion (Greece) earthquake for different ground motion parameters: (**a**) PGA and (**b**) Spectral displacement at the elastic fundamental period (Rossetto and Elnashai 2003)

terise the ground motion but are related to the spectral acceleration or spectral displacement at the fundamental elastic period of vibration (e.g. Rossetto and Elnashai 2003; Scawthorn et al. 1981; Shinozuka et al. 1997). The latter has been an important development as it has meant that the relationship between the frequency content of the ground motion and the fundamental period of vibration of the building stock is taken into consideration; in general this has been found to produce vulnerability curves which show improved correlation between the ground motion input and damage (see Fig. 1).

The introduction of vulnerability curves based on spectral ordinates, rather than PGA or macroseismic intensities, has also certainly been facilitated by the emergence of more and more ground-motion prediction equations in terms of spectral ordinates. The derivation of such equations has certainly encouraged the most recent trend in vulnerability analysis which is towards an analytical description of the capacity of buildings to resist ground motion, which is described in terms of response spectra (either acceleration and/or displacement spectra) (see e.g. Calvi et al. 2006). However, the main criticism applied to analytical methodologies is that they are rarely calibrated or verified using observed damage data. In Italy there is a wealth of observed damage data following the most significant earthquakes of the last 30 years, and thus this would appear to be a perfect source of information from which empirical vulnerability curves could be derived for comparison with analytical vulnerability predictions.

The aim of this paper is therefore to develop observed damage (empirical) vulnerability functions with an approach which will allow the derived curves to be directly compared with analytical vulnerability curves. Many of the shortcomings of empirical vulnerability curves are related to the fact that the vibration characteristics of the buildings are not taken into account (due to the use of a single parameter for the ground intensity such as PGA) or that the macroseismic intensity is used to define the ground shaking, but this parameter is directly obtained from observed damage data and thus the damage and ground shaking intensity are not independent. In order to overcome both aforementioned weaknesses of empirical vulnerability curves, the functions presented herein will consider the displacement demand to each of the building typologies from a ground-motion prediction equation using an estimated mean period of vibration for each building type. An equivalent linearisation approach will be applied (see e.g. Iwan 2002) such that the displacement demand obtained from the secant period of vibration at each damage limit state will be calculated considering a mean limit state ductility for each building class. In this way, the approach used to derive the empirical vulnerability curves will emulate as closely as possible that used in the derivation of the mechanics-based vulnerability curves, leading to what should hopefully be a valid comparison.

2 Italian earthquake damage databases

Over the past 30 years in Italy, a concerted effort has been made to collect detailed observed damage data following significant earthquakes. In the study described herein, the post-earthquake damage surveys from the most important earthquakes that have occurred in Italy have been utilised: Irpinia 1980, Eastern Sicily 1990, Umbria-Marche 1997, Umbria 1998, Pollino 1998 and Molise 2002. The main parameters of these earthquakes are highlighted in Table 1 and Fig. 2 shows the map of the earthquake epicentres and the municipalities which were surveyed following each event.

Table 1 Earthquakes in Italy for which post-earthquake damage surveys are available

Event	Date	Most effected area	Epicentral latitude	Epicentral longitude	Moment magnitude (M_W)
Irpinia 1980	23-Nov-80	Irpinia-Basilicata	40.850	15.280	6.89
Eastern Sicily 1990	13-Dec-90	South eastern Sicily	37.266	15.121	5.68
Umbria-Marche 1997	26-Sept-97	Apennines Umbro-Marchigiano	43.019	12.879	6.05
Umbria 1998	26-Mar-98	Apennines Umbro-Marchigiano	43.252	13.071	5.33
Pollino 1998	09-Sept-98	Apennines Calabro-Lucano	40.038	15.937	5.68
Molise 2002	31-Oct-02	Molise	41.694	14.925	5.78

Gruppo di Lavoro CPTI (2004)

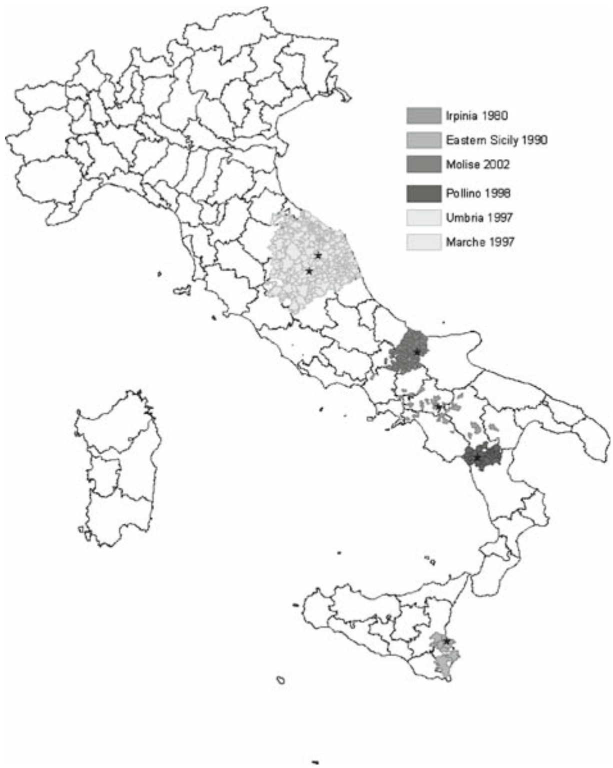

Fig. 2 Map illustrating the earthquake epicentres and the surrounding municipalities which were surveyed

In each survey that was carried out following the aforementioned earthquakes, a different survey form was utilised for the collection of damage data. Hence, the data collected from the different earthquakes are not uniform and in the present study a significant amount of effort was required in order to obtain consistent data which could be analysed together for the derivation of vulnerability curves.

The data have been organised in terms of the damage suffered by the vertical structure. Although it is appreciated that the overall damage to buildings cannot be described using simply the damage to the vertical structure, only this description has been used herein as the analytical methods considered predict only the vertical damage to the structure.

The damage states which have been used are based on the limit state conditions defined in the recent Italian seismic design/assessment regulations (OPCM 2003): slight damage, significant damage and collapse. The slight damage limit state condition refers to the situation where the building can be used after the earthquake without the need for repair and/or strengthening. Beyond the limit condition of significant damage the building cannot be used after the earthquake without strengthening. Furthermore, this level of damage is such that it might not be economically advantageous to repair the building. If the collapse limit condition is achieved, the building becomes unsafe for its occupants as it is not capable of sustaining any further lateral force nor the gravity loads for which it has been designed. In order to relate the damage reported in the survey forms to the damage states described above, it has been necessary to make a number of assumptions; for example in the survey form used for the 1980 Irpinia earthquake, the insignificant and slight damage states were both taken to be slight damage as defined herein, whilst the very severe, partially collapsed and collapsed damage descriptions were all taken to correspond to the collapse damage state used in the current study. This assumption was considered to be valid as the description of severe damage and partially collapsed damage states in this survey form corresponded to buildings which were "to be demolished" and, as such, reconstruction of the building would be necessary and thus the direct cost of these damage states should be similar. Similar hypotheses were required for each of the survey forms which were utilised.

Table 2 summarizes the data available from each database. A common classification scheme, which could be applied to all of the databases, has been identified and as indicted in the table, the data which could be obtained from all the survey forms comprises: ISTAT code, number of storeys above ground, building typology (based on the vertical structure) and damage to the vertical structure. The building typology has been defined considering just the vertical structure as this is how the building typologies have been described in the 1991 Census data, the use of which is described further in Sect. 3.1. The buildings have been classified as reinforced concrete (RC), masonry, and buildings with both RC and bearing masonry walls (referred to as *mixed* in the following). Unfortunately, it has not been possible to derive vulnerability curves for classes of buildings using the information related to the horizontal structure, presence of tie beams, roof type etc. as such information is not available for all databases (see Table 2). Nevertheless, such detailed information on the exposed buildings within a given area is often not available and thus even if it were possible to define vulnerability curves for more detailed classes of buildings, it would most probably be very difficult to assign these curves to the existing building stock in a seismic risk assessment study.

The ISTAT code defines the municipality where the building is located; as will be described further in Sect. 3, this information is then used to estimate the intensity of ground shaking to which the building was subjected during the earthquake. Without this information the building damage data becomes useless and, therefore, any building without an ISTAT code has been removed from the dataset. Furthermore, buildings without information regarding the vertical structural type, the number of storeys and the level of damage have also been

Table 2 Data available for each database (highlighted in bold data was used in the common classification scheme)

		Irpinia 1980	Sicilia Orientale 1990	Marche 1997	Umbria 1997–1998	Pollino 1998	Molise 2002
	Building code	x	x	x	x	x	x
	ISTAT code	**x**	**x**	**x**	**x**	**x**	**x**
	No storeys			x			
	No. of storeys above ground	**x**	**x**	**x**	**x**	**x**	**x**
	No of storeys underground	x		x	x	x	x
	Construction date	x		x	x	x	x
	Date of last retrofit				x	x	x
	Regularity in plan					x	x
	Regularity in elevation					x	x
Vertical structure	**Masonry**	**x**	**x**	**x**	**x**	**x**	**x**
	Reinforced concrete	**x**	**x**	**x**	**x**	**x**	**x**
	Mixed	**x**	**x**	**x**	**x**	**x**	**x**
	Tie rods/Tie beams		x	x		x	x
	Damage to vertical structure	**x**	**x**	**x**	**x**	**x**	**x**
Horizontal structure	Rigid	x	x	x	x	x	x
	Semi-rigid			x		x	x
	Flexible	x	x	x	x	x	x
Roof	Heavy/lightweight	x	x		x	x	x
	With/without retaining ties		x			x	x
	Total no. of buildings	38,079	5462	47,881	64,337–4994	18222	19893
	No. of processed buildings	26,440	2258	29,496	21,676–3144	13353	8285
	% of processed buildings	70%	41%	62%	34%–63%	73%	42%

removed from the database. Hence, due to the absence of the required aforementioned information, a large number of buildings have been disregarded from the database and the sample size has therefore reduced significantly. All together, a database of 104,652 buildings has been processed from an initial database which comprised 198,868 buildings; therefore, only about 50% of the available surveys have been processed. Finally, only 96,282 structures are considered in the development of the vulnerability curves presented herein as the "mixed" buildings (i.e. buildings with both RC and bearing masonry walls) have been disregarded for the time being. This is due to the fact that the ISTAT building Census data (described further in Sect. 3.1) do not take into account mixed structures independently and thus it has not been possible to obtain the total number of buildings of this construction type in each of the surveyed municipalities.

Although only a limited amount of the available data presented in Table 2 (the rows highlighted in bold) were used for the generation of vulnerability curves, the remaining data were used to carry out a brief study regarding the identification of the most vulnerable characteristics of masonry buildings in Italy. In particular, the horizontal structural type (rigid/flexible), the presence or absence of tie rods and tie beams, the regularity in plan and in elevation, and the roof typology (with/without retaining ties) have been studied in greater detail to identify whether there is a strong correlation between these characteristics and the damage attained.

Figure 3 shows the proportion of buildings in each damage band as a function of the properties related to horizontal structural type (Fig. 3a), presence of tie beams/rods (Fig. 3b), retaining ties (Fig. 3c) and regularity in elevation and plan (Fig. 3d). As can be seen from Fig. 3, the most influential parameter appears to be that related to the rigidity of the floor, with the buildings with rigid floors experiencing less damage as compared to those with flexible floors. The lack of tie rods/beams is also seen to cause an increase in the proportion

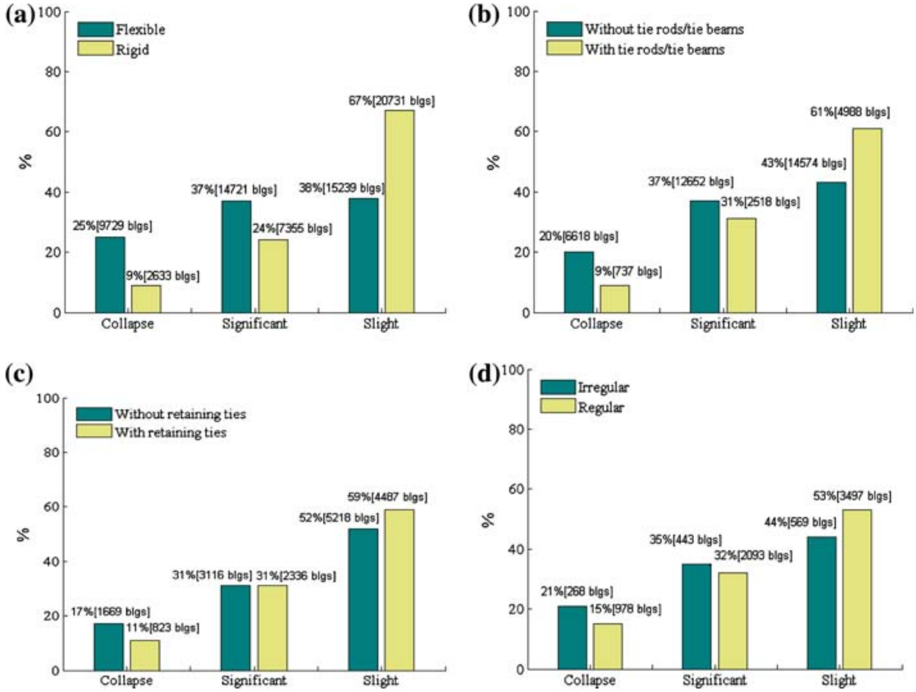

Fig. 3 Influence of (**a**) the horizontal structure, and (**b**) the presence or absence of tie rods and tie beams, (**c**) the presence or absence of retaining ties in the roof and (**d**) the irregularity in plan and elevation, on the level of damage for masonry buildings with 1 and 2 storeys

of damaged buildings, as would be expected, and similarly, irregularity in plan leads to higher proportions of significantly damaged and collapsed buildings. The least influential parameter appears to be the presence or absence of retaining ties, with similar proportions of buildings in each damage band.

It should be noted that there is most certainly a correlation between the parameters considered in Fig. 3; for example, buildings with rigid reinforced concrete floors generally have tie beams whilst the presence of retaining ties has an influence on the damage when the roof structure is pitched, but may otherwise be unimportant. In order to consider the influence of each parameter separately, further subdivisions of the data would be required. Unfortunately this has not been possible with the survey forms used herein as only a limited number of forms have been compiled in detail for all of the parameters considered in Fig. 3. For example, rigid floors may be constructed in reinforced concrete or in steel, yet this distinction has not always been provided in the forms. This is obviously an area where further, more detailed studies are required such that the separate effect of each parameter, the correlation between different parameters and the relationship between damage and ground-motion intensity can be predicted. Nevertheless, despite these shortcomings with the data, the results obtained are in line with the behaviour of masonry buildings under seismic action, as has been observed in other, more detailed studies (e.g. Zuccaro 2004).

Figure 4 presents a summary of the overall damage data which have been reported for each earthquake. It is worth noting that for reinforced concrete structures, slight damage is

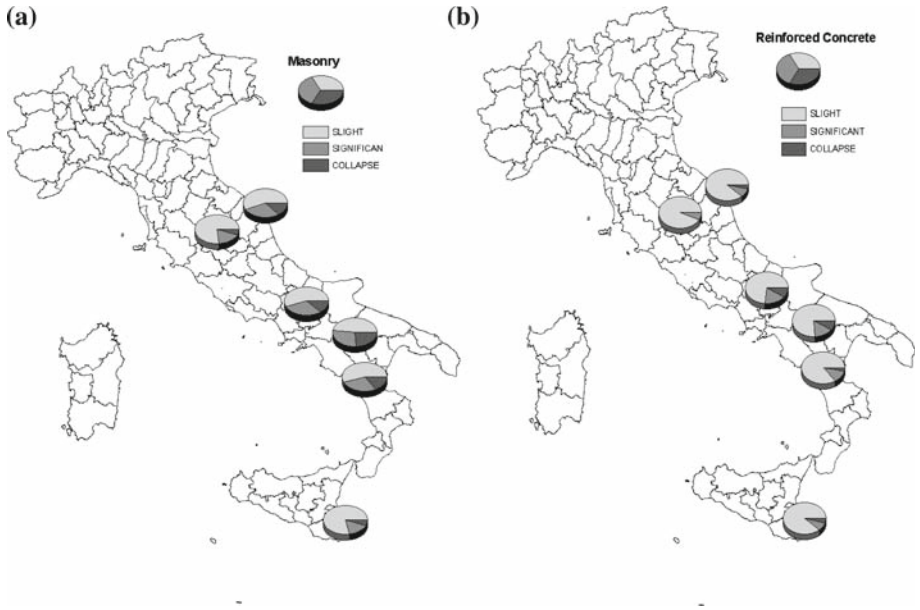

Fig. 4 Damage distribution from each earthquake for (**a**) masonry buildings and (**b**) reinforced concrete buildings

the prevailing damage state whereas for masonry buildings the levels of damage are much more evenly distributed.

3 Empirical vulnerability curves from observed damage data

3.1 Observed damage data

The building dataset used for the generation of empirical vulnerability curves has been described in Sect. 2. The aforementioned dataset mainly includes buildings which suffered damage, though in some cases not all of the buildings within a given region will have been surveyed as the post-earthquake reconnaissance effort is often only carried out on those buildings for which it is explicitly requested (e.g. Zuccaro 2004). It is true that in some cases the municipalities have been entirely surveyed; however, to use only these complete datasets would have reduced considerably the size of the sample. When deriving vulnerability curves the size of the whole population of buildings within the affected region is required in order to calculate the proportion of the whole building stock which has reached or exceeded each damage state. The difference between the whole population of buildings within a given region and the number of surveyed buildings is assumed to represent the number of undamaged buildings.

To obtain the total number of buildings within each municipality, the ISTAT Italian Census of 1991 has been used. The Census data in 1991 was collected in terms of dwellings; however, with the Census form, each dwelling was classified as being located within a building with a certain number of dwellings (from 1 to >30), of a given construction type (RC, RC with pilotis, Masonry, Other), and with a given number of storeys (1–2, 3–5, >5). Hence, based on

Table 3 Summary of the masonry and reinforced concrete (RC) buildings used in the vulnerability study divided into storey classes

	No. of surveyed buildings		No. of processed buildings		No. of buildings registered in the Census of 1991	
	Masonry	RC	Masonry	RC	Masonry	RC
1–2 storey	85,418	5130	67,762	1688	309,823	117,205
3–5 storey	27,491	2804	24,557	1832	84,747	62,536
>5 storey	356	313	226	217	527	4544

the Census forms compiled for all dwellings within each census tract/municipality, Meroni et al. (2000) have estimated the number of buildings classified according to the period of construction, number of storeys and the vertical structural type within each municipality. The errors associated with the use of Census data based on the number of dwellings to arrive at the number of buildings are recognised by the authors and have been identified and quantified in some areas of Italy (see e.g. Frassine and Giovinazzi 2004). However, without the presence of detailed exposure data it is necessary to make some sort of hypothesis in order to obtain the number of buildings of a given construction type and with a given number of storeys.

The inclusion of the period of construction in the Census data has allowed the buildings which were constructed between 1981–1991 to be removed from the database such that the number of buildings at the time of the 1980 Irpinia earthquake could be estimated. However, the buildings which collapsed during the 1980 Irpinia and 1990 Eastern Sicily earthquake were no longer present at the time of the 1991 ISTAT Census, yet they were present at the time of the earthquake, and thus there are problems with the definition of the number of buildings within the municipality when the earthquake occurred. In order to overcome this problem, the number of collapsed buildings in the database from the Irpinia and Eastern Sicily earthquakes have been added to the '91 Census data.

In Table 3, a complete summary of the total number of buildings registered in the Census of 1991, the number of damaged buildings in the database and the number of processed buildings is given for both masonry and reinforced concrete buildings. The data are divided into three different classes of storey: 1–2, 3–5 and >5 storeys. It is noted that a significant proportion of the surveyed buildings are missing from the first two columns of Table 3 as they correspond to those forms for which either the number of storeys, the type of vertical structure, or the damage to the vertical structure was missing. Hence, the difference between the number of processed buildings and the number of surveyed buildings is even larger than that which can be interpreted from Table 3.

The buildings sample is further classified as a function of damage level in Table 4 for both masonry and reinforced concrete buildings. As discussed in Sect. 2, a significant number of structures have been removed from the database because of incompleteness and deficiencies in the survey forms; as seen in Table 4 such problems have effected mainly the reinforced concrete buildings and the high-rise masonry buildings.

3.2 Ground-motion prediction equations

As discussed in the Introduction, vulnerability curves express the probability of reaching or exceeding a given damage state, given a function of the earthquake intensity. In order to estimate the level of ground shaking at each of the municipalities where the damaged buil-

Table 4 Summary of the masonry and reinforced concrete (RC) buildings used in the vulnerability study divided into storey and damage classes

	Damage	Masonry			Reinforced concrete		
		No. of surveyed buildings	No. of processed buildings	Percentage of processed buildings (%)	No of surveyed buildings	No of processed buildings	Percentage of processed buildings (%)
1–2 storey	Collapse	15,245	12,156	80	626	80	13
	significant	26,475	20,548	78	998	226	23
	slight	43,698	35,058	80	3506	1382	39
3–5 storey	Collapse	2954	2540	86	198	97	49
	significant	7708	6728	87	363	204	56
	slight	16,829	15,289	91	2243	1531	68
>5 storey	Collapse	32	14	44	18	10	56
	significant	94	54	57	38	15	39
	slight	230	158	69	257	192	75

dings were surveyed, ground-motion prediction equations in terms of displacement spectral ordinates have been used; displacement spectra have been used as it is well known that there is a strong correlation between observed damage and displacement demand. The prediction equation proposed by Sabetta and Pugliese (1996) and the relationship recently derived by Faccioli et al. (2007) have been considered for the prediction of ground shaking intensity as they have both been used in seismic hazard assessments in Italy.

The Sabetta and Pugliese (1996) equation is representative of the ground shaking expected from Italian earthquakes as it was generated through the regression analysis of analog accelerograms recorded in Italy. The equation provides pseudo-spectral velocity ordinates from 0.04 to 4s as a function of magnitude (moment magnitude M_w was considered herein), epicentral distance and soil conditions; the spectral velocity at 5% damping can be transformed into the spectral displacement via the pseudo-spectral relationships. Faccioli et al. (2007) have developed a ground-motion prediction equation for spectral displacement ordinates up to 15 s by fitting data from a worldwide database of digitally recorded accelerograms of shallow crustal earthquakes; some Italian analog accelerograms from the 1980 Irpinia earthquake were added to the database after a careful scrutiny of their long period characteristics. This equation directly provides the spectral displacement at 5% damping as a function of the magnitude, hypocentral distance and soil conditions. The difference in the maximum response period between the two ground-motion prediction equations is due to the fact that analog accelerograms have been used by Sabetta and Pugliese (1996) for which it is difficult to obtain an accurate prediction of the displacement at large periods (when double integration of the acceleration is carried out, errors are accumulated for the low frequency contents of the ground motion).

A comparison between the median displacement spectra with 5% damping predicted with the two relationships is reported in Fig. 5 for the Irpinia 1980 earthquake ($M_s = 6.9$) for the municipality of Paternopoli (which is at an epicentral distance of 25 km). The Irpinia earthquake has been chosen as recordings from this earthquake were used in the regression

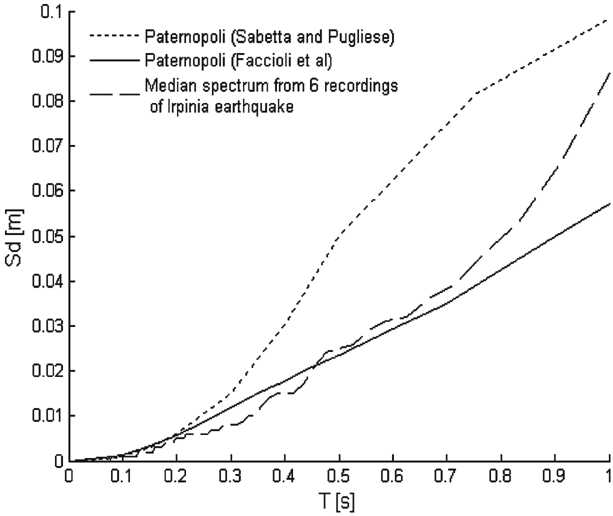

Fig. 5 Comparison between median displacement spectra evaluated with the equation proposed by Sabetta and Pugliese (1996), the equation proposed by Faccioli et al. (2007) and the median spectrum from a number of recordings of the Irpinia earthquake

of both aforementioned ground-motion prediction equations. A median spectrum based on the spectra from 6 recordings of this earthquake, which have a mean epicentral distance of 25 km and are on rock and stiff soil, has also been plotted in Fig. 5. The median spectrum from the recordings of this earthquake can be seen to be closer to the median spectrum obtained using the Faccioli et al. (2007) ground-motion prediction equation, at least in the low period range. At higher periods, the median spectrum from the recordings is slightly closer to the Sabetta and Pugliese equation, though it should be noted that this comparison cannot be considered a validation or otherwise of either equation, considering the reduced number of recordings involved.

From Fig. 5 it is also possible to observe how the displacements predicted by Faccioli et al. (2007) are considerably lower than those given by Sabetta and Pugliese (1996) and this highlights the large epistemic uncertainty which is present in the prediction of the ground shaking intensity for the derivation of vulnerability curves. There is also, of course, a component of aleatory variability in the ground-motion prediction equations which further increases the uncertainty in the prediction of the ground shaking experienced by the damaged buildings. However, the use of the median ground motion may be justified as the damage data from many sites at similar distances from the epicentre are combined in the generation of the vulnerability curves (as discussed in Sect. 3.4) and it is expected that in some of the municipalities the ground motions were higher than the median and in others they were lower than the median (due to the intra-event variability).

For the vulnerability curves derived herein, the ground-motion prediction equation proposed by Faccioli et al. (2007) has been utilised; however, some preliminary studies have been carried out using the Sabetta and Pugliese (1996) equation and it has been seen that the epistemic uncertainty in the ground motion prediction equation has a large influence on the vulnerability curves due to the very different spectral displacement values which are predicted for a given building class. Further uncertainties arise due to the site conditions in each of the municipalities which are currently unknown and thus, as discussed in the next section, average site conditions have been considered. Only the results based on the Faccioli et al. (2007) equation will be presented herein as this prediction equation has been directly developed in terms of spectral displacement and the data used for the regression analysis of this equation were from digital records, considered to be more reliable in terms of displacement prediction, but which unfortunately were not available when the Sabetta and Pugliese (1996) relationship was developed.

3.3 Calculation of limit state displacement demand for each structural type

The ground shaking intensity in terms of spectral displacement, S_d, for each building class within a given municipality (i.e. for a certain epicentral/hypocentral distance and site condition) for a given earthquake scenario (i.e. for a certain M_W) has been calculated using the Faccioli et al. (2007) ground motion prediction equation using the mean period of vibration of the surveyed buildings, which is related to structural type, level of damage and number of storeys, as will be discussed below. As the site conditions within each of the municipalities are unknown at the locations of the damaged buildings, average soil mechanical characteristics have been assumed in the ground-motion prediction equation: 50% site B (with a shear wave velocity, V_{s30}, between $360\,\mathrm{m\,s^{-1}}$ and $800\,\mathrm{m\,s^{-1}}$), and 50% site C (with $180\,\mathrm{m\,s^{-1}} \leq V_{s30} \leq 360\,\mathrm{m\,s^{-1}}$).

To calculate the fundamental period of vibration of each structural type for a given number of storeys, a random building population has been generated using the characteristics of the Italian building stock. A simulated design procedure has been applied to each random building and then a simplified pushover analysis has been carried out, as fully described in Borzi et al. (2008a). This methodology, SP-BELA (Simplified Pushover-Based Earthquake Loss Assessment) is one of the mechanics-based methods with which the empirical vulnerability curves are compared and thus the aim is to ensure that the displacement demand in both the empirical methodology and the mechanics-based methodologies are similarly predicted. The form of the simplified pushover curve for the single-degree-of-freedom (SDOF) representation of the building is presented in Fig. 6a; this curve illustrates the lateral capacity of the building (collapse multiplier, λ) and the displacement capacity (Δ) at the three different limit states to damage from which the mean ductility at the second and third limit states can be estimated. The ranges of mean ductility values calculated for both the masonry and reinforced concrete buildings are reported in Table 5.

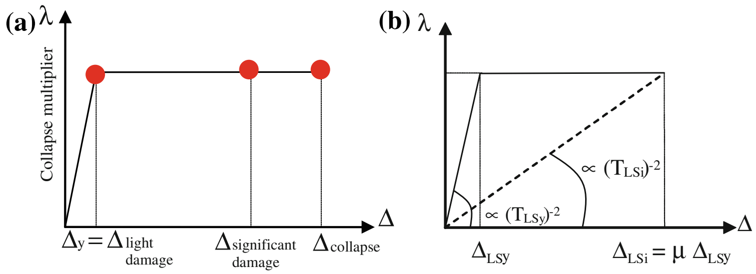

Fig. 6 (**a**) Simplified pushover curve (collapse multiplier vs. displacement) illustrating the limits to each damage state. (**b**) Relationship between elastic and secant period of vibration for limit state *i*

Table 5 Summary of the mean ductility values calculated for the masonry and reinforced concrete (RC) buildings

	Masonry		Reinforced concrete	
	μ_2	μ_3	μ_2	μ_3
1–2 storey	2.6	5.3	2.0	2.8
3–5 storey	1.6	2.6	1.7	2.0
>5 storey	–	–	1.6	1.9

The fundamental period of vibration (at the first limit state, T_y) of each of the random buildings in the population is calculated using the following formula:

$$T_y = 2\pi\sqrt{\frac{\Delta_y}{g\lambda}} \tag{3.1}$$

The mean fundamental period of vibration of the masonry buildings was found to vary between 0.2 s and 0.4 s for low-rise and between 0.4 and 0.8 s for mid-rise. For reinforced concrete buildings the mean period of vibration for the low-rise buildings ranged between 0.5 s and 0.9 s and for mid-rise it was seen to vary between 1.2 s and 1.7 s. The period of vibration at the higher damage states is calculated considering an equivalent linearisation approach based on the secant stiffness at the limit state under consideration and hence the relationship between the elastic period of vibration and that corresponding to a limit state *i* is given by the following relationship (which can be deduced from Fig. 6b):

$$T_{LSi} = T_{LSy}\sqrt{\mu_{LSi}} \tag{3.2}$$

The equivalent linearisation approach presented in Fig. 6b assumes the structure behaves linearly, whereas in reality structures have a non-linear hysteretic behaviour. To account for this energy dissipation, a displacement reduction factor η is introduced to reduce the displacement demand associated with the damaged buildings. The reduction factor can be related to the equivalent viscous damping, ξ_{eq}, through the following expression, as presented in EC8 (CEN 2003):

$$\eta = \sqrt{\frac{10}{5+\xi_{eq}}} \tag{3.3}$$

In order to calculate the equivalent viscous damping ξ_{eq}, the recent expressions proposed by Dwairi et al. (2007) have been used. For masonry buildings, the expression which relates to the *Small Takeda Hysteretic System* has been assumed due to the reduced energy dissipation in masonry structures, as opposed to concrete structures:

$$\begin{aligned} \xi_{eq} &= \xi_v + C_{ST}\left(\frac{\mu-1}{\pi\mu}\right)\% \\ C_{ST} &= 50 + 40\left(1-T_{eq}\right) \quad T_{eq} < 1\,\text{sec} \\ C_{ST} &= 50 \quad\quad\quad\quad\quad\;\; T_{eq} \geq 1\,\text{sec} \end{aligned} \tag{3.4}$$

In Eq. 3.4, ζ_v represents the damping ratio characterising the elastic response, commonly assumed to be 5%, μ represents the ductility of the system and T_{eq} is the equivalent/secant period of vibration. The expression relating to the *Large Takeda Hysteretic System* (Dwairi et al. 2007) has been used for reinforced concrete structures:

$$\xi_{eq} = \xi_v + C_{LT}\left(\frac{\mu-1}{\pi\mu}\right)\%$$

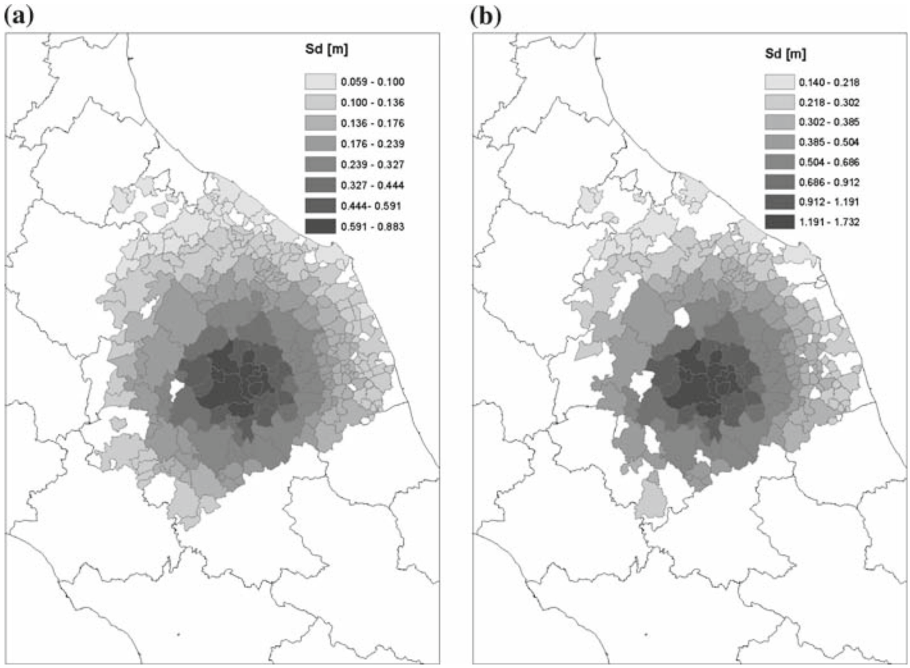

Fig. 7 Predicted spectral ordinates S_d (metres) in each municipality for 2 storey masonry buildings for the (**a**) slight damage and (**b**) collapse damage states

$$\begin{aligned} C_{LT} &= 65 + 50(1 - \mathrm{T_{eq}}) \quad \mathrm{T_{eq}} < 1\,\mathrm{sec} \\ C_{LT} &= 65 \qquad\qquad\qquad\quad \mathrm{T_{eq}} \geq 1\,\mathrm{sec} \end{aligned} \tag{3.5}$$

Hence, once the mean period of vibration of the building class has been calculated according to the height, structural typology and level of damage, an overdamped spectral displacement value can be associated to each building of the database based on the earthquake magnitude, hypocentral distance of the municipality within which the buildings are located, the soil conditions and the spectral reduction factor (Eq. 3.3). Discrete bins of imposed displacement demand have been considered for each limit state such that the number of damaged buildings within each range would be statistically significant. As an illustrative example, the predicted spectral ordinates S_d in each of the surveyed municipalities after the event of Umbria-Marche 1997 are reported in Fig. 7; these spectral ordinates represent the displacement demand to the 2 storey masonry buildings within each of the municipalities at the first and third limit states.

From Fig. 7 it can be seen that the number of municipalities reduces from the slight to the collapse damage states as the displacement demand has only been calculated for those municipalities where a given damage state was observed within the building stock.

3.4 Generation of vulnerability curves

Once the spectral displacements for a given building type (i.e. construction type and number of storeys) have been calculated for the three levels of damage, as illustrated in Fig. 7, the number of buildings of that building class in a given damage state within each displacement

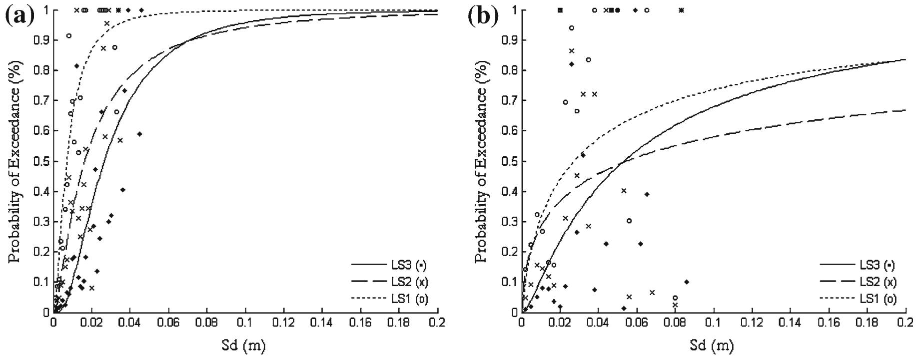

Fig. 8 Vulnerability curves for masonry buildings (**a**) 1–2 storey (**b**) 3–5 storey. LS1: slight damage, LS2: significant damage and LS3: collapse

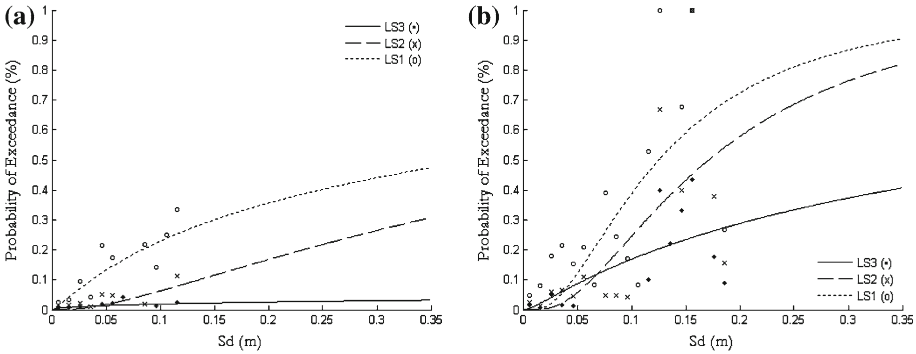

Fig. 9 Vulnerability curves for reinforced concrete buildings (**a**) 1–2 storey (**b**) 3–5 storey. LS1: slight damage, LS2: significant damage and LS3: collapse

range for all of the earthquakes has been integrated. For each building class, the number of buildings which reach a given damage state (none, slight, significant and collapse) can then be found and divided by the number of buildings within all the municipalities which were subject to the considered displacement range. The probability of reaching each limit state is then used to calculate the probability of exceedance, as required for the generation of vulnerability curves. Figures 8 and 9 show the produced vulnerability curves obtained using this procedure for masonry and reinforced concrete buildings, respectively. Vulnerability curves for buildings with more than 5 storeys have not been developed herein as there are very few data for this class (see Table 3) and the majority of the surveyed structures were collapsed and thus it would not be possible to generate vulnerability curves for all three limit states.

The vulnerability curves for the three limit states can be seen in some of the previous figures to cross; this is obviously not physically possible but arises from the fact that much of the observed data collected on the damaged buildings were not utilised. Hence, there are some ranges of displacement within which data for only one limit state are available. Some sensitivity analyses have been run to observe the influence of the size of the bins of spectral displacement on the generated curves as this has an influence on the damage data which are used for each range of spectral displacement; although a change in the bin size was seen to increase the number of limit states which are represented in some displacement ranges, in others there was a reduction in the number of limit states. The influence of changing the bin size was thus seen to have a negligible influence on the crossing vulnerability curves.

4 Comparison of empirical and analytical vulnerability curves

In this section a comparison of the derived vulnerability curves (Figs. 8, 9) with those produced by two different mechanics-based methodologies is presented. The mechanics-based methodologies which have been considered are those which are being developed by the authors: DBELA, Displacement-Based Earthquake Loss Assessment (Crowley et al. 2004) and SP-BELA (Borzi et al. 2008a,b).

For what concerns reinforced concrete buildings, DBELA considers two different types of response mechanisms and thus two distinct vulnerability curves. For the purpose of the current study only one response mechanism has been considered: the column-sway (soft-storey) mechanism, as this is assumed to represent the behaviour of the surveyed buildings which are mainly non-seismically designed. For the SP-BELA methodology, the aforementioned assumption on the collapse mechanism is not required. As mentioned previously, in SP-BELA the buildings are designed and their capacity is then calculated and the collapse mechanism is checked; the buildings for the current study have been designed for gravity-loads only. In both methodologies, a random building population is simulated where each building has a randomly defined set of structural characteristics (material properties, geometrical properties etc.) and the period of vibration and displacement capacity of each building at each of the three limit states is calculated.

In both methods, the bare frame reinforced concrete buildings have been considered because the displacement capacity is mainly ruled by the frames and not by the infill panels, which govern, on the other hand, the building stiffness. This latter characteristic only has a limited influence on the mechanics-based vulnerability curves derived herein as the displacement demand (S_d) is an imposed parameter and thus does not depend on the vibration characteristics of the buildings. For each level of displacement demand, the number of randomly generated buildings which have a limit state displacement capacity lower then the displacement demand are counted and the probability of exceeding the limit state is thus obtained.

In Figs. 10 and 11, the observed and mechanics-based vulnerability curves are compared for masonry and reinforced concrete buildings, respectively. Only a limited number of comparisons have been reported as it is felt that only those related to the most statistically significant data, as presented previously in Table 4, are valid for comparison purposes. Hence for masonry buildings only the low-rise slight damage and collapse limit state curves are reported whilst for reinforced concrete the mid-rise slight and significant damage curves have been compared. As can be deduced from Figs. 10, 11, the vulnerability curves produced by the mechanics-based methodologies are more conservative than the curves fit to the observed damage data.

The main factors which contribute to the differences in the vulnerability predictions between the analytical and empirical methodologies are as follows:

- The resistance provided by non-structural elements (e.g. partition walls) play a role in terms of the overall building performance which is not taken into account in the numerical models considered herein. Furthermore, the period of vibration of the reinforced concrete buildings used to calculate the displacement demand in the empirical vulnerability curves (see Sect. 3.3) currently relates to bare frames and thus it is expected that, at least for the slight damage state, the displacement demands have been overestimated.

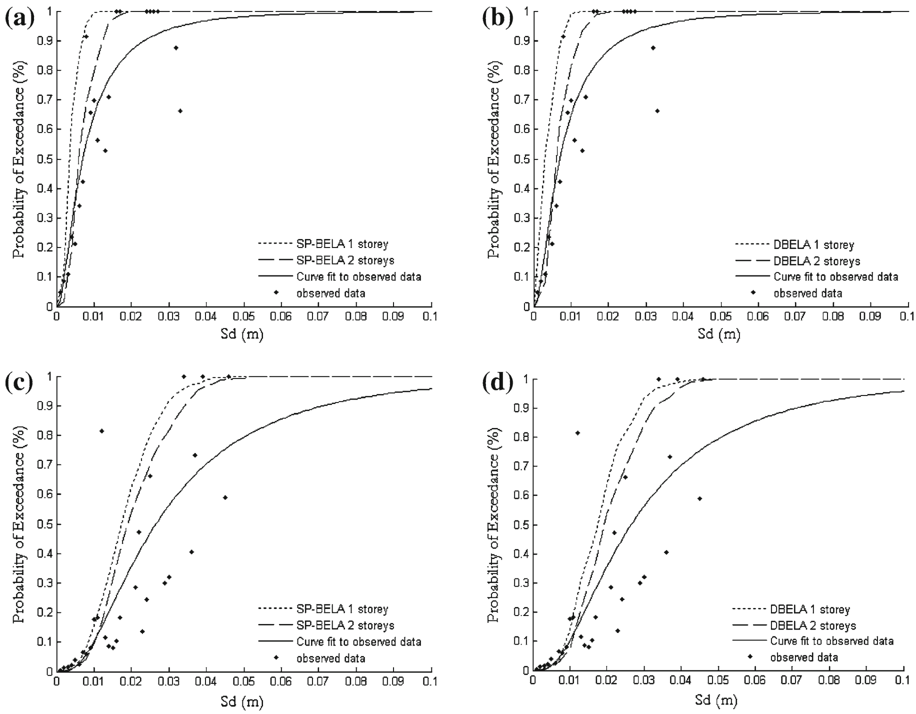

Fig. 10 Comparison between vulnerability curves fit to observed data and curves produced by mechanics based methodologies for masonry buildings with 1–2 storeys with slight damage: (**a**) SP-BELA method (**b**) DBELA method, and collapse: (**c**) SP-BELA method (**d**) DBELA method

- As a consequence of the incomplete compilation of the survey forms, or due to compilation errors, about 50% of the surveyed buildings have been removed from the dataset of observed data. Hence, there is surely an underestimation of the number of damaged buildings. Since the building type, number of storeys or location of these surveyed buildings cannot be identified, it has been impossible to implement an equivalent reduction in the total number of buildings within each municipality which is used to calculate the proportion of buildings exceeding each damage state.

However, it is worth noting that the comparison between the vulnerability curves is relatively satisfactory for masonry buildings with 1–2 storeys. As a matter of fact, the sample size of this class of masonry buildings is the most statistically significant; the size of the observed data is about 70% of the entire database. On the other hand, the comparison is very poor for reinforced concrete buildings which, however, correspond to only about 4% of the total database and therefore, for this structural type, the curves fit to the observed data are not statistically significant. Moreover, as reported in Tables 3 and 4, around 80% of the surveyed low-rise masonry buildings have been processed for the limit states presented in Fig. 10 whilst only about 60% of the surveyed mid-rise reinforced concrete structures have been utilised for the limit states presented in Fig. 11.

5 Discussion and conclusions

During the post-processing of the observed damage data presented herein, a number of difficulties and unresolved issues have arisen. These difficulties mainly relate to: the uncertainty

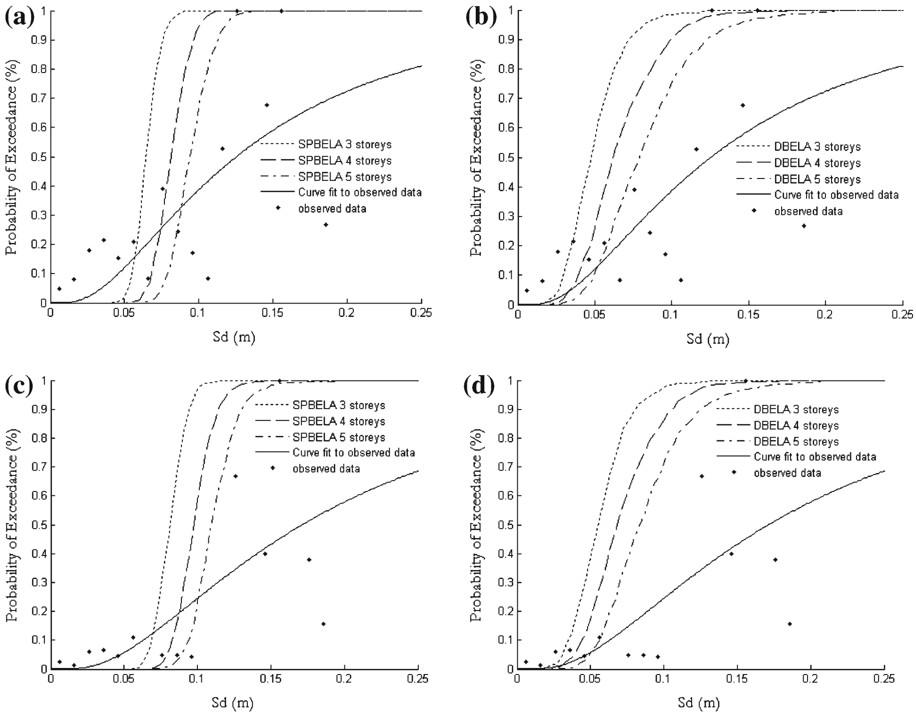

Fig. 11 Comparison between vulnerability curves fit to observed data and curves produced by mechanics based methodologies for reinforced concrete buildings with 3–5 storeys with slight damage: (**a**) SP-BELA method (**b**) DBELA method, and significant damage: (**c**) SP-BELA method (**d**) DBELA method

in predicting the ground shaking to which the damaged buildings were subjected; the uncertainty in the exposure data which is required to define the number of buildings of each building class in each municipality; and, perhaps most importantly, the fact that 50% of the survey forms have been removed from the database.

The median overdamped spectral displacement based on the Faccioli et al. (2007) ground-motion prediction equation for B/C type ground conditions has been used to predict the ground shaking in each municipality. Without recordings from the sites where the damaged buildings were surveyed, there is a large amount of uncertainty in the level of the ground shaking assigned to the damage data in the generation of the vulnerability curves and this is due to the epistemic uncertainty in the choice of ground-motion prediction equation, the aleatory variability in the ground-motion prediction equation, and the unknown site conditions within each municipality. However, as mentioned previously, as the damage data from many sites at a similar distance from the epicentre are combined in the calculation of the probability of exceedance, it is consistent to use the median ground motion that is expected at these sites.

Another source of uncertainty has certainly been introduced through the use of the ISTAT 1991 Census data to define the total number of buildings in each municipality of each building class (i.e. construction type and number of storeys) as these data have been obtained from the number of dwellings. Recent studies have shown that the number of buildings generated from the '91 Census dwelling data overestimates the actual number of buildings in the city of Catania by around 10–20% (Frassine and Giovinazzi 2004). Furthermore, the '91 ISTAT data considered the buildings used for residential purposes only, whilst in the available databases the surveyed damaged structures were used for various purposes: residential, industrial, commercial, etc. Another problem which has arisen with the ‘91 ISTAT data is that the structures built after 1991 are not included and thus for the events considered in this report which occurred after 1991 (of which there are four), the size of the building sample that was utilised has probably been underestimated.

The incompleteness and deficiencies in the survey forms and the errors produced in the computerisation of the data led to a notable reduction of the database during post-processing. About 50% of the surveyed buildings were removed from the dataset because fundamental information was missing: the ISTAT code (required to define the location of the building and thus ground shaking intensity), the number of storeys, or the construction type. The initial database was comprised of 198,868 buildings, out of which only 93,911 have been processed for the presented curves. This factor is probably the most influential of all of those discussed herein as it means that the vulnerability curves derived in Sect. 3.4 are underestimated, especially for the reinforced concrete buildings and the mid-rise masonry buildings. Only those related to low-rise masonry buildings have been based on a significant proportion of the collected damage data.

To improve the vulnerability curves presented herein it will therefore be necessary to improve the estimation of the exposure at the time of the earthquake and to complete the observational database in order to ensure all the information on the surveyed buildings can be processed. It is unlikely that this will be something which can be carried out in a the short-term and thus, in the meantime, the authors will continue to develop the methodologies used to produce the presented mechanics-based vulnerability functions. A positive outcome of this study has been that the mechanics-based vulnerability curves for low-rise unreinforced masonry buildings compared well with the observed damage data and, considering that the majority of the processed data related to this building class, this result could be construed as a preliminary validation of these mechanics-based methodologies.

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