PART II: VULNERABILITY ASSESSMENT

CHAPTER 6
VULNERABILITY ASSESSMENT OF DWELLING BUILDINGS

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6.1. Introduction

Risk is defined as the potential economic, social and environmental consequences of hazardous events that may occur in a specified area unit and period of time. Its estimation requires a multidisciplinary approach that takes into account not only the expected physical damage understood as the damage suffered by structures, the number and type of casualties or the economic losses, but also social, organizational and institutional factors. At urban level, for example, vulnerability should be also related to the social fragility and the lack of resilience of the exposed community, that is, to its capacity to absorb the impact and control its implications. Nevertheless, a holistic approach to estimate risk aiming to guide the decision making at urban level should start with the evaluation of physical damage scenarios as an essential tool, because they are the result of the convolution between hazard and physical vulnerability for buildings and infrastructure (Cardona, 2001; Barbat, 2003). Accordingly, the evaluation of physical vulnerability and risk of buildings is the main purpose of this chapter. Some definitions related to these concepts are introduced here below.

Risk, $R\mathrm{ie}|_T$, can be defined as the probability of loss in an exposed element e as a consequence of the occurrence of an event with intensity larger than or equal to i during an exposition period T.

Hazard, $H\mathrm{i}|_T$, can be understood as the probability of occurrence of an event with an intensity greater than or equal to i during an exposition period T.

Vulnerability, $V\mathrm{e}$, is the intrinsic predisposition of the exposed element e to be affected or of being susceptible to suffer a loss as a result of the occurrence of an event with intensity i.

Starting from these definitions, risk is a function f of the convolution between hazard $H\mathrm{i}$ and vulnerability $V\mathrm{e}$ during an exposition period T

$$R\mathrm{ie}|_T = f\,(H\mathrm{i} \otimes V\mathrm{e})|_T \tag{6.1}$$

where the symbol ⊗ stays for convolution (Cardona and Barbat, 2000).

The major part of losses due to earthquakes has its origin in the deficient seismic behaviour of structures. In spite of the advances of research in earthquake engineering in general and particularly on seismic design codes, catastrophic losses have occurred recently in many countries in the world, including countries in which earthquake engineering studies are priority tasks. It is clear that new developments in earthquake resistant design can only be applied to new projects, which represent a small part of the existing structures in a seismic area. Therefore, the only possibility of reducing earthquake losses is improvement of the seismic behaviour of existing structures. The aim of risk studies is to predict the expected damage in structures due to a specified

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earthquake. A seismic risk analysis addressed to earthquake emergency management and protection strategies planning, requires territorial scale evaluation. Once the expected damage is predicted, it is possible to find solutions to diminish it, which rebound in the cost of the structures; this cost can be compared with the expected losses, in order to decide if structural retrofit or structural reinforcement are feasible. In spite of the importance of this type of studies, standard methodologies to estimate the vulnerability of structures are not available.

6.2. Methodologies for vulnerability assessment

Dolce et al., (1994) classified methodologies for the evaluation of structural vulnerability in four groups: (a) direct, which assesses in a simple way the damage caused in a structure by a given earthquake; (b) indirect, which determines first a vulnerability index of the structure and then assesses the relationship between damage and seismic intensity; (c) conventional, which is essentially a heuristic method, introducing a vulnerability index independent of the damage prediction; (d) hybrid, which combines elements of the previous methods with expert judgments. The selection of one of these methods depends on the objectives of the study, the type of the results required and on the available information. On the other hand, fragility functions, damage probability matrices and vulnerability functions obtained from observed structural damages during past earthquakes in a seismic area were the preferred tools in seismic risk studies performed in the past (Kappos et al., 1995; Benedetti and Petrini, 1984; Barbat et al., 1996).

The damage probability matrices and vulnerability functions are defined in the following way: 1) Damage Probability Matrices (DPM) express in a discrete form the conditional probability $P[D = j|i]$ to obtain a damage level j, due to an earthquake of severity i (Whitman 1974); and 2) Vulnerability Functions are relations expressing the vulnerability in a continuous form, as functions of certain parameters that describe the size of the earthquake (Benedetti and Petrini 1984). The vulnerability assessment is performed in terms of qualitative parameters: buildings are classified in vulnerability classes, and a DPM is assigned to each class or, alternatively, scores are attributed to the buildings considering their typological, structural, geometric and constructive characteristics; a simple model is then defined as a function of the evaluated scores relating the seismic input to the expected damage (Benedetti and Petrini, 1984; FEMA, 1998).

A complete observed damage data base would be desirable for applying such approaches; however, this is only possible in certain high seismicity areas where post-earthquake survey studies have been properly performed. Where the information is limited, damage matrices and vulnerability functions can be established using the available data and local expert opinion (Anagnos et al., 1995). Finally, in countries without any available damage data base, the information obtained in other similar areas is applied in a direct (Chavez and García-Rubio, 1995) or modified form, using expert judgment (Bustamante et al., 1995). Some authors have used hybrid methodologies to assess the vulnerability of buildings (Chavez and García-Rubio, 1995), developing fragility curves and damage probability matrices in order to estimate the feasibility of seismic rehabilitation of existing reinforced concrete (RC) buildings.

As the available data are often incomplete and do not concern all the building typologies and all the intensities that would be necessary to be represented in a model, probabilistic processing of the observed data is supported or completely replaced by other approaches such as structural analysis methods (Milutinovic and Trendafiloski, 2003), expert judgement (ATC-13, 1985), neural network systems (Dong et al., 1988) or fuzzy set theory (Sanchez-Silva and Garcia, 2001). To complete the undesirable lack of earthquake damage information in an area, simulation procedures can be also applied. The probabilistic analysis of computer generated structural responses obtained by using complex or simplified models and nonlinear analysis procedures of representative buildings can provide fragility curves, damage probability matrices and vulnerability functions relating seismic intensity or peak ground acceleration with damage (Nocevski and Petrovski, 1994; Kappos et al., 1995; Singhal and Kiremidjian, 1996). In these studies, the damage estimated for a generic structure pertaining to a given typology is considered as representative for the whole range of structures belonging to the mentioned structural typology.

In the vulnerability index method, the study is extended to a large number of classes of buildings within each of the considered typologies; these classes are defined through parameters which cover most of the structural characteristics, aiming to discriminate among different seismic behaviours of buildings with the same structural typology located in a specified seismic area (Benedetti and Petrini, 1984). This method, based on a great amount of damage survey data corresponding to several seismic zones of Italy, identifies the most important eleven parameters controlling the damage in buildings caused by earthquakes and qualifies them individually by means of qualification coefficients K_i affected by weights W_i which try to emphasize their relative importance. The method makes an overall qualification of the buildings by means of a vulnerability index Iv. Thus, the global vulnerability index of each building is evaluated by means of the formula

$$I\mathrm{v} = \sum_{i=1}^{11} K_i \times W_i \tag{6.2}$$

Using vulnerability functions, it is possible to relate Iv with an overall damage index D of the buildings, whose values, expressed as a percentage, also range between 0 and 100. The eleven mentioned parameters are: structural system organization, structural system quality, conventional strength, retaining walls and foundation, floor system, configuration in plant, configuration in elevation, maximum distance between walls, roof type, non-structural elements and preservation state.

An economic damage index corresponding to the physical risk of buildings could be defined as the relation between the damage repair cost and reposition cost. Both in the case of unreinforced masonry buildings and reinforced concrete buildings it is not reasonable to consider the overall structural damage index equal to the economic damage index, due to the presence of non- structural elements (architectural elements, equipment, installations, etc.) which usually contribute to the major part of the economic losses (Gunturi, 1993).

A damage index can be obtained for each structural component of a reinforced concrete building. Then it is possible to evaluate an economic damage index for each floor $D_{\mathrm{ec},k}$ by means of the equation

$$D_{\mathrm{ec},k} = \frac{\sum D_{\mathrm{ec},i} \times w_i}{\sum w_i} \tag{6.3}$$

where $D_{\mathrm{ec},i}$ is the structural damage index for each element i of floor k and w_i is the reposition cost of this element. The economic damage index of the entire structure can be then obtained as the average of all the structural floor indices. The economic floor damage index for architectural elements and equipment can be also evaluated starting from the maximum drifts and accelerations of floors obtained from a nonlinear analysis of the structures. Finally, the global economic damage index of the building can be obtained as a weighted average of the economic structural and non-structural economic damage indices.

In the United States, and nowadays also in Europe, the most recent trends in the field of vulnerability evaluation for risk analysis operate with simplified mechanical models, essentially based on the Capacity Spectrum Method (Freeman, 1998b; NIBS, 1997, 1999 and 2002). This method permits evaluating the expected seismic performance of structures by comparing, in spectral coordinates (Sd, Sa), their seismic capacity with the seismic demand, described by Acceleration-Displacement Response Spectra (ADRS) adequately reduced in order to take into account the inelastic behaviour (Fajfar, 2000). For purposes of territorial vulnerability assessment, capacity spectrum procedures do not necessarily use capacity curves obtained by pushover analyses, but they ascribe bilinear capacity curves defined by yielding (Dy, Ay) and ultimate (Du, Au) capacity points to each building typology; these curves vary depending on geometrical and technological parameters, characteristic of the buildings (number of floors, code level, material strength, drift capacity, etc.). Such an approach provides reliable results if applied to a built-up area characterized by a typological building homogeneity and by consolidated seismic design codes. This is not the case in the European Union where seismic codes are very different and where various typologies of masonry buildings can be distinguished in the territory. In this case, the employment of capacity based methods needs, yet, a robust experimental validation, at least on the traditional masonry constructions; for this reason, statistical methods based on damage observations are required.

A method derived in a theoretically rigorous way, starting from EMS-98 macroseismic scale (Grünthal, 1998) definitions overcomes the distinction between typological and rating methods and allows carrying out a vulnerability analysis with a single approach, graded to different levels according to the quantity and quality of the available data and the size of the territory. The method, which is applicable to all the European regions, has been verified on the basis of data observed after earthquakes occurred in different countries. The vulnerability index method in its version mentioned before and the capacity spectrum method are described in detail in the following sections of this chapter.

6.3. Vulnerability index method based on the EMS-98 macroseismic scale

6.3.1. EMS-98 BASED VULNERABILITY CURVES

6.3.1.1. *Vulnerability model implicitly contained in the EMS-98 scale*
The aim of a macroseismic scale is to measure the earthquake severity starting from the observation of the damage suffered by buildings and therefore it represents, for forecast purposes, a vulnerability model able to supply, for a given intensity, the probable damage distribution. In this sense, the EMS-98 scale, which will probably be used in the future at European level, contains a clear and detailed definition of the different building typologies and a precise description of the degrees of damage and of the damage distribution related to each degree of intensity. It makes reference to vulnerability classes, which are a way to group together buildings characterized by a similar seismic behaviour; six classes (from A to F) of decreasing vulnerability are introduced and, for each of them, the intensity, that can be estimated from a certain damage pattern, is supplied in terms of damage matrices. The damage matrix defined in the EMS-98 scale, which considers 5 damage grades and also the absence of damage, gives the probability that buildings belonging to a certain vulnerability class suffer a certain damage level for a given seismic intensity (see the example given in Table 6.1).

These damage matrices can be used for vulnerability assessments, but the model that they provide is vague and incomplete. The definition for the damage quantification in Table 6.1 is, indeed, provided in a vague way through the quantitative terms “Few”, “Many”, “Most” as the aim is a post-earthquake survey and a precise determination of quantities is not envisaged. Moreover, the distribution of damage is incomplete as the macroseismic scale only considers the most common and easily observable situations (for example, no information is provided for damage grade 3, 4 and 5 for I = VI and vulnerability class B in Table 6.1).

Table 6.1. Damage model provided by the EMS-98 scale for classes of vulnerability B (Milutinovic and Trendafiloski, 2003; Giovinazzi and Lagomarsino, 2004)

Class B					
Damage level	1	2	3	4	5
Intensity					
V	Few				
VI	Many	Few			
VII		Many	Few		
VIII			Many	Few	
IX				Many	Few
X					Many
XI					Most
XII					

6.3.1.2. *The incompleteness problem*
In order to solve the incompleteness problem, the damage distributions of earthquakes occurred in the past has been considered; the idea is to complete the EMS-98 model

introducing a proper discrete probability distribution of the damage grade. The binomial distribution could be a possibility as it has been successfully used for the statistical analysis of data collected after the 1980 Irpinia earthquake (Italy) (Braga et al., 1983); but the simplicity of this distribution, which depends on only one parameter, does not allow defining the scatter of the damage grades around the mean value.

Sandi and Floricel (1995) observed that the dispersion of the binomial distribution is too high, when you consider a detailed building classification; this may lead to overestimating the number of buildings suffering serious damages, in the case of rather low values of the mean damage grade. The distribution that better suits the specific requirements is the beta distribution (also employed in ATC-13, 1985):

$$PDF : p_\beta(x) = \frac{\Gamma(t)}{\Gamma(r)\Gamma(t-r)} \frac{(x-a)^{r-1}(b-x)^{t-r-1}}{(b-a)^{t-1}} \qquad a \le x < b \tag{6.4}$$

$$\mu_x = a + \frac{r}{t}(b-a) \tag{6.5}$$

where *a, b, t* and *r* are the parameters of the distribution; μ_x is the mean value of the continuous variable *x*, which ranges between *a* and *b* and Γ(r) is the gamma function.

In order to use the beta distribution, it is necessary to make reference to the damage grade *D*, which is a discrete variable (5 damage grades plus the absence of damage); for this purpose, it is advisable to assign value 0 to the parameter *a* and value 6 to the parameter *b*. Starting from this assumption, it is possible to calculate the probability associated with damage grade *k* (*k*=0, 1, 2, 3, 4, 5) as follows:

$$p_k = P_\beta(k+1) - P_\beta(k) \tag{6.6}$$

Following from this assumption, the mean damage grade, defined as the mean value of the discrete distribution:

$$\mu_D = \sum_{k=0}^{5} p_k \cdot k \tag{6.7}$$

The mean value μ_x (6.5) can be correlated through the following third degree polynomial:

$$\mu_x = 0.042\mu_D^3 - 0.315\mu_D^2 + 1.725\mu_D \tag{6.8}$$

Thus, by using (6.5) and (6.8), it is possible to correlate the two parameters of the beta distribution with the mean damage grade

$$r = t(0.007\mu_D^3 - 0.0525\mu_D^2 + 0.2875\mu_D) \tag{6.9}$$

The parameter t affects the scatter of the distribution; if t=8 is used, the beta distribution looks very similar to the binomial distribution.

6.3.1.3. *The vagueness problem*

Once the problem of incompleteness is solved by using the discrete beta distribution, it is necessary to tackle the problem of the vagueness of the qualitative definitions (few, many, most) in order to derive numerical DPM for EMS-98 vulnerability classes. As translation of the linguistic terms into a precise probability value is arbitrary, they can be better modelled as bounded probability ranges. Fuzzy sets theory (often proposed for seismic risk assessment methods) has offered an interesting solution to the problem, leading to the estimation of upper and lower bounds of the expected damage (Bernardini, 1999). According to fuzzy sets theory, the qualitative definitions can be interpreted through membership functions χ (Dubois and Parade, 1980). A membership function defines the belonging of single values of a certain parameter to a specific set; the value of χ is 1 when the degree of belonging is plausible (that is to say almost sure), while a membership between 0 and 1 indicates that the value of the parameter is rare but possible; if χ is 0, the parameter does not belong to the set.

Figure 6.1 shows the range of percentage of damaged buildings corresponding to the quantitative terms given by the EMS-98. It contends that while there are some definite ranges (few, less then 10%; many, 20% to 50%; most, more than 60%), there are situations of different terms overlapping (between 10% and 20% can be defined as both few and many; 50% and 60%, both many or most). These qualitative definitions are represented through the membership functions χ in Figure 6.1.

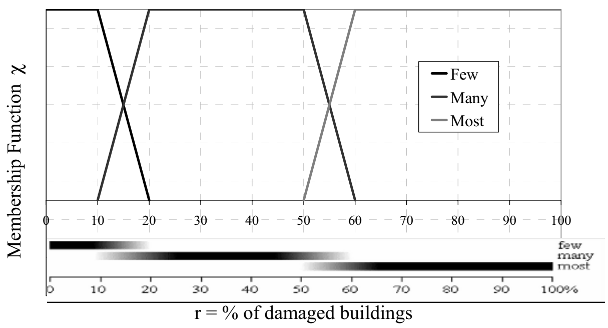

Fig. 6.1. Percentage ranges and membership functions χ of the quantitative terms "Few", "Many", "Most"

6.3.1.4. *EMS-98 damage probability matrices*

Using fuzzy sets theory and starting from EMS-98 definitions (e.g. Table 6.1), it is possible to build a DPM through the discrete beta distribution. Recalling that to each value of parameter μ_D having definitely assumed a=0, b=6 and for a fixed value of t, corresponds a fixed damage grade distribution, researchers have looked for μ_D values able to represent the terms "Few", "Many", "Most" in a plausible and then in a possible way, according to the membership functions associated to the quantitative definitions. In order to make the operation easier, a value t=8 may be used, but it has been verified that for different values of t, the differences observed are negligible. From the probabilistic distributions corresponding to the computed μ_D values, the percentages of damage have been attributed to the different damage grades. As an example, it is possible to consider the vulnerability class B and the macroseismic intensity VI. Table 6.2 shows for the vulnerability class B, the upper and lower bounds of the mean damage grade related to plausibility and possibility. The corresponding distributions of the damage grades are also shown; the dark and light grey cells correspond to the control definitions and the value that determines the bound is shown as a bold character (Giovinazzi and Lagomarsino, 2004).

Table 6.2. Damage distributions and mean damage values related to the upper and lower bounds of plausibility and possibility ranges for vulnerability class B

Class B						
Damage level	1	2	3	4	5	
Intensity VI	Many	Few				μ_D
B^+ Upper plausible	32.0	**10**	1.9	0.2	0.0	0.68
B^- Lower plausible	**20**	4.3	0.6	0.0	0.0	0.43
B^{++} Upper possible	40.6	**20**	5.5	0.7	0.0	1.81
B^{--} Lower possible	**10**	1.6	0.2	0.0	0.0	0.25

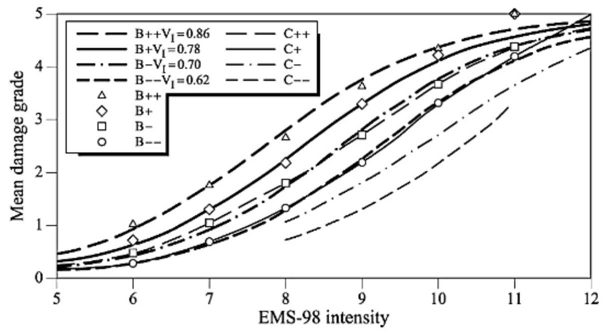

Fig. 6.2. Class B and C plausibility and possibility curves and their interpolation

Repeating this procedure for each vulnerability class and for the different intensity degrees, it is possible to obtain, point by point, the plausible and possible bounds of the mean damage. Connecting these points, draft curves are drawn, which define the plausibility and possibility areas for each vulnerability class, as a function of the macroseismic intensity (see Figure 6.2).

6.3.1.5. *Vulnerability index and vulnerability curves*

Observing the curves of Figure 6.2, it stands out that there is a plausible area for each vulnerability class and intermediate possible areas for contiguous classes. In other words, the area between B+ and B- is distinctive for class B, while there is a contiguous area in which the best buildings of class B and the worst of class C coexist (the B- curve coincides with the C++ one; the B-- curve coincides with the C+ one). Another important observation is that curves in Figure 6.2 are, more or less, parallel because the damage produced to buildings of a given vulnerability class by an earthquake of certain intensity, is the same as that caused by an earthquake with the next intensity degree to buildings of the subsequent vulnerability class. On the basis of these considerations, a conventional Vulnerability Index V_I is introduced within the frame of the fuzzy set theory, indicating that a building pertains to a vulnerability class. The numerical values of the vulnerability index are arbitrary as they are only scores which quantify, in a conventional way, the seismic behaviour of a building (they are a measure of the weakness of a building to resist earthquakes). For the sake of simplicity, a 0 to 1 range has been chosen, allowing all possible behaviours to be covered. The values close to 1 correspond to the most vulnerable buildings and the values close to 0 to high-code designed structures. Thus, the membership of a building to a specific vulnerability class can be defined by means of this vulnerability index (see Figure 6.3); in compliance with the fuzzy set theory they have a plausible range (χ =1) and linear possible ranges, representative of the transition between two adjacent classes.

According to the fuzzy definition of the vulnerability index, Table 6.3 shows the most probable value for each vulnerability class V_I^{c*}, the bounds V_I^{c-} V_I^{c+} of the uncertainty range and the upper and the lower bound of the possible values V_{Imax}^{c} and V_{Imin}^{c}. It must be noticed, according to Figure 6.3, that the partition of the fuzzy field is not restricted to the minimum value of –0.02 and to the maximum value of 1.02; actually it is not possible to keep out of the evaluation buildings weaker than those of class A or better designed than those of class F.

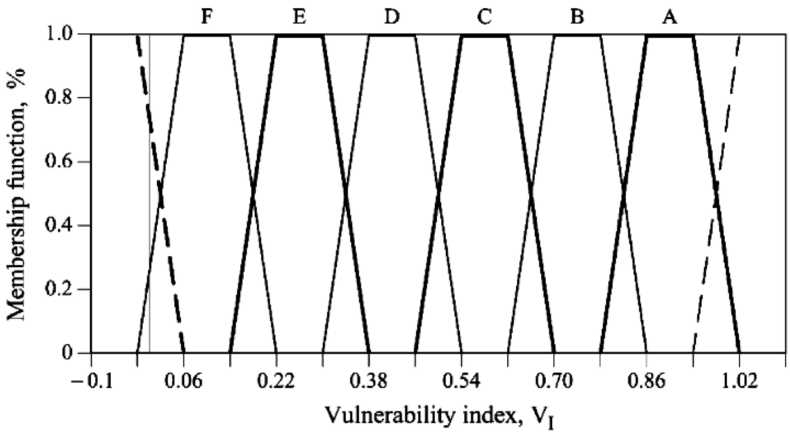

Fig. 6.3. Vulnerability index membership functions for the EMS-98 vulnerability classes

For the operational implementation of the methodology, it is particularly useful to define an analytical expression, capable of describing the curves in Figure 6.2; therefore, the mean damage grade μ_D is expressed by means of the following function depending on the macroseismic intensity *I* and the vulnerability index V_I:

$$\mu_D = 2.5 \left[1 + tanh\left(\frac{I + 6.25 \cdot V_I - 13.1}{2.3}\right)\right] \quad (6.10)$$

Table 6.3. Vulnerability index values for the different vulnerability classes

	$V_{I\,min}^{c}$	V_I^{c-}	V_I^{c*}	V_I^{c+}	$V_{I\,max}^{c}$			$V_{I\,min}^{c}$	V_I^{c-}	V_I^{c*}	V_I^{c+}	$V_{I\,max}^{c}$
A	1.02	0.94	0.9	0.86	0.78		D	0.54	0.46	0.42	0.38	0.3
B	0.86	0.78	0.74	0.7	0.62		E	0.38	0.3	0.26	0.22	0.14
C	0.7	0.62	0.58	0.54	0.46		F	0.22	0.14	0.1	0.06	-1.02

6.3.2. EVALUATION OF THE VULNERABILITY INDEX

The EMS-98 macroseismic scale contains a table with a typological classification of buildings representative for the European countries and vulnerability table (Table 6.4), which distinguishes the buildings as functions of the structural material: masonry, reinforced concrete, steel and timber. Different buildings having the same structural typology are characterized by a prevailing seismic vulnerability class; nevertheless, it is possible to find buildings with a better or worse seismic behaviour within the same vulnerability class, depending on their design, constructive or structural characteristics. Therefore, the EMS-98 scale subdivides the seismic behaviour of the buildings in six vulnerability classes for which damage probability matrices and vulnerability curves have been evaluated.

The idea highlighted by the EMS-98 scale, according to which the seismic behaviour of a building not only depends on the behaviour of its structural system but also on other factors, has suggested the following definition of the vulnerability index:

$$\overline{V_I} = V_I^* + \Delta V_R + \Delta V_m \tag{6.11}$$

where ΔV_R and ΔV_m are, respectively, a factor of regional type and of behaviour type.

6.3.2.1. *Typological vulnerability index*

EMS-98 table describes the possibility of a given building typology belonging to a vulnerability class through linguistic terms, as it can be seen in Table 6.4: “Most probable class”, “Possible class”, “Unlikely class”. Even in this case, the fuzzy set theory can provide a useful contribution for the linguistic term interpretation. The belonging of each typology to the vulnerability classes is represented in a fuzzy way, by discriminating the most likely class (χ =1), the possible class (χ =0.6) and the unlikely class (χ =0.2) of Table 6.4. It is possible to define the membership function of each building type as a linear combination of the vulnerability class membership functions, each one considered with its own degree of belonging. As an example, the membership function for massive stone masonry buildings (M4) is shown in Figure 6.4 and it is defined as:

$$\chi_{M4}(V_I) = \chi_C(V_I) + 0.6 \cdot \chi_B(V_I) + 0.2 \cdot \chi_D(V_I) \tag{6.12}$$

where χ_C, $\cdot\chi_B$ and $\cdot\chi_D$ are defined in Figure 6.3 (see also Table 6.4.).

From the membership function of each typology, five representative values of V_I have been defined (see Figure 6.4) through a defuzzification process (Ross, 1995): the most probable value of the typological vulnerability index V_I^* for a specific building type is calculated as the centroid of the membership function. V_I^- and V_I^+, evaluated by a 0.5-cut of the membership function, are the bounds of the uncertainty range of V_I^* for that building type. V_{Imin} and V_{Imax} correspond to the upper and lower bounds of the possible values of $\bar{V}_{\mathrm{I}}$, that is the final vulnerability index value, for the specific building type. Whatever is the estimated amount of the behaviour modifiers and the regional factor, the final vulnerability index has to comply with this possibility range.

Table 6.4. Vulnerability classes of different building typologies (Building Typology Matrix, BTM)

Typologies		Building type	Vulnerability Classes A	B	C	D	E	F
Masonry	M1	Rubble stone	Most probable					
	M2	Adobe (earth bricks)	Most probable	Possible				
	M3	Simple stone	Unlikely	Most probable				
	M4	Massive stone		Possible	Most probable	Unlikely		
	M5	Unreinforced M (old bricks)	Unlikely	Most probable	Unlikely			
	M6	Unreinforced M with r.c. floors		Possible	Most probable	Unlikely		
	M7	Reinforced or confined masonry			Unlikely	Most probable	Possible	
Reinforced Concrete	RC1	Frame in r.c. (without ERD)	Unlikely	Possible	Most probable	Unlikely		
	RC2	Frame in r.c. (moderate ERD)		Unlikely	Possible	Most probable	Possible	
	RC3	Frame in r.c. (high ERD)			Unlikely	Possible	Most probable	Possible
	RC4	Shear walls (without ERD)		Unlikely	Most probable	Possible		
	RC5	Shear walls (moderate ERD)			Unlikely	Most probable		
	RC6	Shear walls (high ERD)				Unlikely	Most probable	Possible
Steel	S	Steel structures			Unlikely	Possible	Most probable	Possible
Timber	W	Timber structures		Unlikely	Possible	Most probable	Possible	
Situations:		Most probable class	Possible class	Unlikely class				

ERD – "Earthquake Resistance Design"

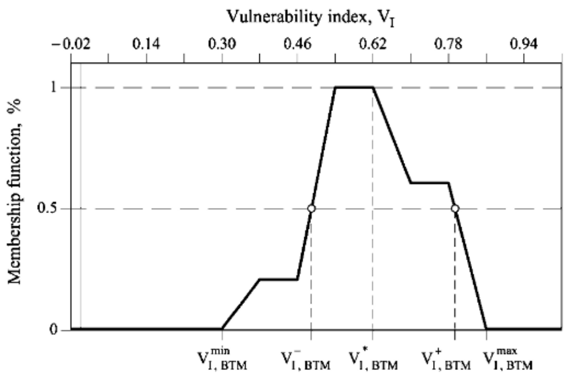

Fig. 6.4. Vulnerability Index membership functions for M4 massive stone typology and V_{I} values

$$\max\left(\bar{V}_{\mathrm{I}};V_{\mathrm{I\,min}}\right)\leq\bar{V}_{\mathrm{I}}\leq\min\left(\bar{V}_{\mathrm{I}};V_{\mathrm{I\,max}}\right) \qquad (6.13)$$

These values are represented in Figure 6.4 for the massive stone masonry typology and in Table 6.5 for all the EMS-98 buildings typologies.

Table 6.5. Vulnerability index values for the EMS-98 building typologies

Typologies		Building type	Vulnerability classes				
			$V_{I\,min}$	V_I^-	V_I^*	V_I^+	V_{Imax}
Masonry	M1	Rubble stone	0.62	0.81	0.873	0.98	1.02
	M2	Adobe (earth bricks)	0.62	0.687	0.84	0.98	1.02
	M3	Simple stone	0.46	0.65	0.74	0.83	1.02
	M4	Massive stone	0.3	0.49	0.616	0.793	0.86
	M5	Unreinforced M (old bricks)	0.46	0.65	0.74	0.83	1.02
	M6	Unreinforced M with r.c. floors	0.3	0.49	0.616	0.79	0.86
	M7	Reinforced or confined masonry	0.14	0.33	0.451	0.633	0.7
Reinforced Concrete	RC1	Frame in r.c. (without ERD)	0.3	0.49	0.644	0.8	1.02
	RC2	Frame in r.c. (moderate ERD)	0.14	0.33	0.484	0.64	0.86
	RC3	Frame in r.c. (high ERD)	-0.02	0.17	0.324	0.48	0.7
	RC4	Shear walls (without ERD)	0.3	0.367	0.544	0.67	0.86
	RC5	Shear walls (moderate ERD)	0.14	0.21	0.384	0.51	0.7
	RC6	Shear walls (high ERD)	-0.02	0.047	0.224	0.35	0.54
Steel	S	Steel structures	-0.02	0.17	0.324	0.48	0.7
Timber	W	Timber structures	0.14	0.207	0.447	0.64	0.86

ERD – "Earthquake Resistance Design"

6.3.2.2. *The behaviour modifier factor*

The typological vulnerability index V_I^* calculated for each building typology has to be increased or decreased according to the vulnerability recognized for a certain building. The overall score that modifies the characteristic vulnerability index V_I^* can be evaluated, for a single building, simply summing all the modifier scores.

$$\Delta V_m = \sum V_m \tag{6.14}$$

These modifiers are related to the state of preservation of the buildings, the structural system, the height of the building within each building typology, irregularities in plan, elevation and of stiffness, retrofitting interventions, soil morphology and foundation, as well as aggregate building position and elevation (Milutinovic and Trendafiloski, 2003).

If a group of buildings, belonging to a certain typology, is considered, the modifier factor ΔV_m, is evaluated as:

$$\Delta V_m = \sum_k r_k \cdot V_{m,k} \tag{6.15}$$

where r_k is the ratio of building affected by the behaviour modifier k characterized by a $V_{m,k}$ score. Making reference to single buildings, the behaviour modifier factor ΔV_m is simply the sum of the scores $V_{m,k}$ for the recognized behaviour modifiers. The identification of the behaviour modifiers can be made empirically, based on the observation of typical damage pattern and taking also into account the suggestions made in several inspection forms (ATC-21, 1988; Benedetti and Petrini, 1984; UNDP/UNIDO, 1985) and by previous proposals (Coburn and Spence, 2002). The modifying scores V_m can be also assigned using expert judgment followed by a partial calibration by comparison with other vulnerability evaluations; but a better calibration is desirable on the basis of damage and vulnerability data collected after earthquakes.

Giovinazzi and Lagomarsino (2004) propose behaviour modifier factors and the corresponding scores for masonry and reinforced concrete buildings.

6.3.2.3. *The regional vulnerability factor*
The range bounded by V_I^-, V_I^+ is quite large in order to be representative for the huge variety of the constructive techniques used all around the different countries. The regional vulnerability factor takes into account the characteristics of the buildings belonging to a certain typology at regional level: a major or minor vulnerability could be indeed recognized due to some traditional constructive techniques used in the region.

According to this regional vulnerability factor the V_I^* typological vulnerability index is modified on the basis of an expert judgment or on the basis of the historical data available. The first case is achieved when precise technological, structural and constructive information is available, attesting an effective better or worse average behaviour with regard to the one proposed in Table 6.5. The second one occurs when observed damages data are available; the average curve ($\overline{V}_I = V_I^*$ in Equation 6.11) can be shifted in order to obtain a better approximation for the same data.

6.3.2.4. *Uncertainty range in the vulnerability assessment*
The uncertainties affecting a seismic risk analysis are both epistemic and random. The epistemic ones refer in this case to uncertainties associated with the classification of the exposed building stock into a vulnerability class or into a building typology and by the uncertainties associated with the assignment of a characteristic behaviour to the vulnerability class or building typology (Spence et al., 2003). Considering both kinds of uncertainties allows obtaining the most probable vulnerability index as well as its plausibility and possibility ranges for each vulnerability class (Table 6.3) and for each building typology (Table 6.5).

It must be noticed that the uncertainty affecting building typologies is higher than the one affecting vulnerability classes because the building typology behaviour has been deduced from the one observed from vulnerability classes and, furthermore, because with few data it is more difficult to classify a building into a typology rather than into a vulnerability class. But the knowledge of information additional to the typological one, limits the uncertainties of the building behaviour; it is therefore advisable not only to modify the most probable value V_I^* (according to Equation 6.11), but also to reduce the range of its uncertainty ($V_I^- \div V_I^+$). This goal is achieved by modifying the membership function through a filter function *f*, centred in the new most probable value $\overline{V}_I$, depending on the parameter ΔV_f, representing the width of the filter function (Giovinazzi and Lagomarsino, 2004).

6.3.2.5. *Example*
The city of Barcelona, Spain, is located in an area of low to moderate seismic hazard, but its buildings show a high vulnerability and, consequently, a significant probability of being damaged even in the case of a not excessively severe earthquake. The total number of dwellings of Barcelona is about 700,000, with an average of 2.2 inhabitants in each, and about 63,000 buildings. The majority of Barcelona's most representative unreinforced masonry buildings, with an average age of 60 years, were designed only

considering vertical static loads, without any seismic design criteria, greatly influencing the overall seismic vulnerability of the city. Additionally, some of its particular features, typical for the constructive techniques of the city at that time, have been identified as potential damage sources. The slabs of these unreinforced masonry buildings are made of wood, steel or reinforced concrete, according to the building period, and have ceramic ceiling vaults in all the cases. Due to the higher height of the first floor, almost all of these buildings have soft first floors. In many cases, cast iron columns were used instead of masonry walls at the base and ground floors, reducing thus even more their stiffness. The majority of the reinforced concrete buildings of Barcelona have waffle slabs, a structural member not adequate for seismic areas. Many of the buildings in Barcelona are part of aggregates.

Traditionally, the vulnerability index method identifies the existing building typologies within the studied area and defines their vulnerability class (Table 6.4). For each vulnerability class, the relationship between intensity and damage may be defined by using Damage Probability Matrices. The specific buildings of Barcelona are classified in different groups characterized by a similar seismic behaviour. All the buildings belonging to each typology are cast within the most probable class.

Table 6.6. Vulnerability index for typologies and periods of construction of Barcelona, according to the seismic design level

Periods	**Period of construction**	**Spanish Code**	**Obligation in Barcelona**	**Lateral bracing in constructive practice**	**Seismic Design level**	**Buildings (%)**	Vulnerability Index (V_i)		
							M31 M32 M33	M34	RC32
I	<1950	-----	-----	Absent	Pre-code	50.69	**0.938**	--	--
II	1950-1962	-----	-----	Deficient	Pre-code	17.30	**0.875**	--	--
III	1963-1968	Recommendation MV-101 (1962)	No specified	Deficient	Pre-code	10.91	**0.813**	**0.750**	**0.750**
IV	1969-1974	Seismic code P.G.S.-1 (1968)	Yes	Acceptable	Low-code	9.80	**0.750**	**0.625**	**0.625**
V	1975-1994	Seismic code P.D.S. (1974)	Yes	Acceptable	Low-code	11.07	**0.688**	**0.563**	**0.500**
VI	1995 until now	Seismic code NCSE-94 (1995)	No	Acceptable	Low-code	0.23	**0.688**	**0.563**	**0.500**

Vulnerability indices are assigned to the most representative building typologies of Barcelona, representing scores that quantify their seismic behaviour. A first refinement of this average initial vulnerability index is performed by taking into account the age of the building. The building stock is grouped in 6 age categories by considering reasonable time periods as functions of the existence of seismic codes in Spain and its level, as well as other specific construction features (see Table 6.6) (Lantada et al., 2004).

Figure 6.5 shows vulnerability maps for both the unreinforced masonry and reinforced concrete buildings of Barcelona.

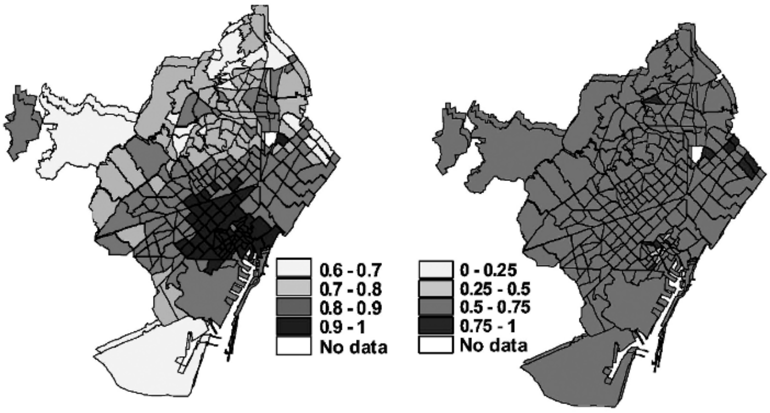

Fig. 6.5. Vulnerability indices maps for unreinforced masonry buildings (left) and reinforced concrete buildings (right)

The global vulnerability index of each building is now evaluated by applying behaviour modifiers (eq. 6.14), which are different for isolated and aggregate buildings. For isolated buildings, the following 4 modifiers were considered: number of floors, irregularity in height, length of the façade and state of preservation. For building in aggregates the effects due to the different heights of adjacent buildings and the effects due to the position of the building in the aggregate (i.e. corner, header, or intermediate) have been taken into account.

6.4. Capacity spectrum method

The capacity spectrum method uses the capacity and demand spectra to obtain the performance point of the structure which corresponds to its maximum spectral displacement, and uses fragility curves to obtain the damage probability for the expected seismic input. Capacity curves are force-displacement diagrams of the structure which correspond to the first mode maximum response of buildings and governing the structural damage; they mainly depend on the structural design and construction practice. The performance of a building is directly influenced by the level and frequency content of the seismic action which controls the peak building response levels. The seismic input is modelled by means of the demand spectrum, which is the inelastic structural response spectrum. This demand spectrum can be obtained by using a nonlinear structural analysis or, in a simplified way, starting from the 5% damped building-site specific elastic response spectrum modified to account for the inelastic structural behaviour. Both the capacity and demand spectra are represented in the spectral acceleration (Sa)-Spectral displacement (Sd) domain.

Fragility curves define the probability that the expected damage d exceeds a given damage state d_S, as a function of a parameter quantifying the severity of the seismic action. Thus, fragility curves are completely defined by plotting $P(d \geq d_S)$ in ordinate and the spectral displacement Sd in abscissa. If it is assumed that fragility curves follow a lognormal probability distribution, they can be completely defined by only two parameters which, in this case, are the mean spectral displacement $\overline{Sd}_{ds}$ and the corresponding standard deviation β_{ds}.

Fragility curves can be obtained in a simplified way starting from the bilinear representation of capacity curves (see Figure 6.6 and Table 6.7). Crossing demand and capacity spectra, the performance point is established and thus the expected spectral displacement which, together with the corresponding fragility curves, allows obtaining probability matrices for the damage scenario corresponding to earthquakes defined by their demand spectra. Therefore, all the fragility and damage analyses are based in a straightforward manner on capacity and demand spectra and fragility curves.

The method for analyzing the seismic damage considers 5 damage states: none, slight, moderate, extensive and complete. For a given damage state, a fragility curve is well described by the following lognormal probability density function:

$$P[ds / Sd] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{Sd}{\overline{Sd}_{ds}}\right)\right] \tag{6.16}$$

where $\overline{Sd}_{ds}$ is the threshold spectral displacement at which the probability of the damage state d_S is 50%, β_{ds} is the standard deviation of the natural logarithm of this spectral displacement, Φ is the standard normal cumulative distribution function and Sd is the spectral displacement. Figure 6.6 and Table 6.7 show how the $\overline{Sd}_{ds}$ thresholds are obtained from the capacity spectra. Concerning β_{ds}, it is well known that the expected seismic damage in buildings follows a binomial probability distribution. Therefore, it is assumed that at the $\overline{Sd}_{ds}$ threshold, the probability of this damage state is 50% and then the probabilities of the remaining damage states are estimated.

Starting from the spectral displacement corresponding to the performance point, damage probability matrices can be obtained by using the corresponding fragility curves. A weighted average damage index, DS_m, can be calculated by using the following equation:

$$DS_m = \sum_{i=0}^{4} DS_i P[DS_i] \tag{6.17}$$

where DS_i takes the values 0, 1, 2, 3 and 4 for the damage states i considered in the analysis and P[DSi] are the corresponding probabilities.

Table 6.7. Damage state thresholds defined in agreement with the capacity spectrum

$\overline{Sd}_1 = 0.7D_y$	Slight
$\overline{Sd}_2 = D_y$	Moderate
$\overline{Sd}_3 = D_y + 0.25(D_u - D_y)$	Extensive
$\overline{Sd}_4 = D_u$	Complete

t can be considered that DS_m is close to the most likely damage state of the structure. According to Equation 6.17, a value DS_m=1.3, for example, indicates that the most probable damage state of a building ranges between *slight* and *moderate*, being more probable the s*light* damage state. This average damage index permits plotting seismic damage scenarios by using a single parameter. Of course, alternative maps may plot the spatial distribution of the probability of occurrence of a specific damage state, that is $P[DS_i]$.

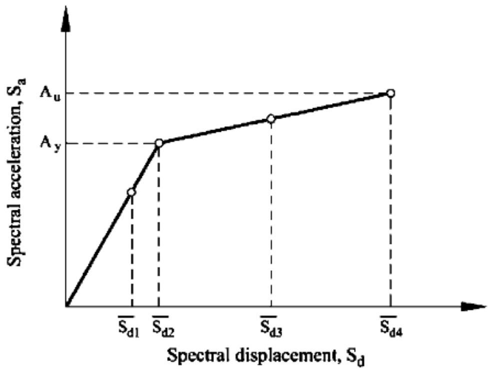

Fig. 6.6. Damage state thresholds from capacity spectrum

6.4.1. EXAMPLE

For illustration of the method we use the same example of Barcelona which has a moderate seismicity and weak tectonic motions; its seismic hazard has been recently re-evaluated defining the action in terms of elastic response acceleration spectra both from a deterministic and a probabilistic approach (Irizarry et al., 2003). Two earthquake scenarios have been developed and used to perform the simulations of seismic risk scenarios, one deterministic, based on a historical earthquake that occurred quite far from the city and whose intensity at the basement and outcrop has been estimated, and the other probabilistic, corresponding to a 475 years return period. The result of both simulations can be seen in Figure 6.7 in acceleration-displacement format (ADRS). The same figure shows a seismic risk scenario in macroseismic intensities, used to develop seismic risk scenarios according to the vulnerability index method.

Detailed structural plans have been used to model representative buildings for low-rise (two stories, 5.2 m high) mid-rise (five stories, 15.8 m high) and high-rise (eight stories, 24.0 m high) reinforced concrete buildings. Capacity curves were obtained by performing non-linear static analyses using RUAUMOKO-2D program (Carr, 2000). In a similar way, based on detailed structural plans, two stories (low-rise), four stories (mid-rise) and six stories (high-rise) buildings of the Eixample district of Barcelona have been modelled. TreMuri program (Galasco et al., 2002) was used to perform the dynamic analyses of the buildings. Pushover analyses allowed obtaining the capacity curves for each building class. Table 6.8 shows the yield and ultimate capacity points defining the bilinear capacity spectra for reinforced concrete and masonry buildings. It can be seen how the capacity decreases with the height of the building both for masonry and for RC buildings.

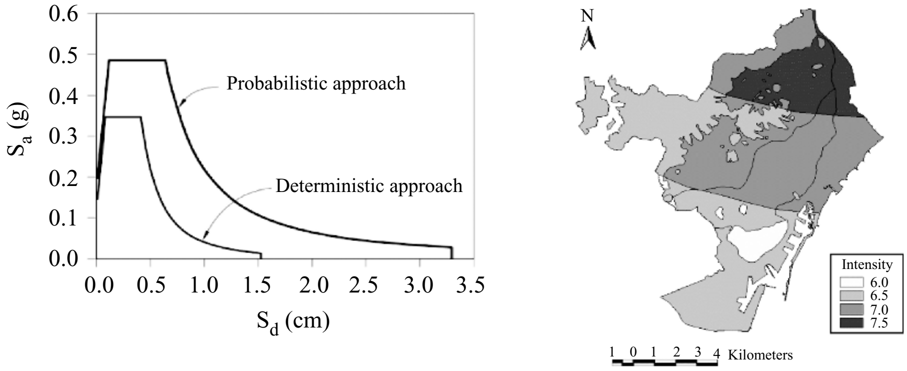

Fig. 6.7. Response spectra for deterministic and probabilistic hazard scenarios (left) and deterministic seismic hazard scenario including local soil effects in intensity (right) (Irizarry et al., 2003)

Table 6.9 shows the expected probabilities of all the damage states when a particular damage state probability is fixed to 50% and a binomial or equivalent beta probability distribution is assumed. In this table, the damage states are represented by numbers from 1 to 4 for damage states *slight* to c*omplete*, respectively. The probabilities in this table are cumulative and correspond to the points shown in Figure 6.8. Parameter DS_m controls the assumed probability distribution. Finally, the function expressed by Equation 6.16 is fitted to the obtained points by means of a least square criterion.

Table 6.8. Yield and ultimate capacity for reinforced concrete (RC) and masonry (M) buildings

Building class	**Yield capacity**		**Ultimate capacity**	
	Dy (cm)	**Ay (g)**	**Du (cm)**	**Au (g)**
Low-rise, RC	0.70	0.129	5.240	0.138
Mid-rise, RC	1.418	0.083	5.107	0.117
High-rise, RC	1.894	0.059	4.675	0.079
Low-rise, M	0.27	0.651	1.36	0.558
Mid-rise, M	0.63	0.133	2.91	0.117
High-rise, M	0.68	0.105	2.61	0.079

Table 6.9. Probabilities of the expected damage states when fixing a 50% probability for each damage state: 1-slight, 2-moderate, 3-extensive and 4-complete

Condition	DS_m	P_β **(1)**	P_β **(2)**	P_β **(3)**	P_β **(4)**
P_β **(1)=0.5**	0.911	**0.500**	0.119	0.012	0.00
P_β **(2)=0.5**	1.919	0.896	**0.500**	0.135	0.008
P_β **(3)=0.5**	3.081	0.992	0.866	**0.500**	0.104
P_β **(4)=0.5**	4.089	1.000	0.988	0.881	**0.500**

Figure 6.8 shows an example of such a fit. Points in this figure correspond to the damage state probabilities and lines are the fitted fragility curves. This figure corresponds to the mid-rise reinforced concrete building class. Table 6.10 shows the corresponding parameters, namely $\overline{Sd}_i$ and β_i, where $i=1,\ldots,4$ defines the fragility curves corresponding to reinforced concrete (RC) and unreinforced masonry (M) building classes. The demand spectra, together with the capacity spectra have been used to obtain the performance point which defines the maximum expected spectral displacement related to a specific demand. Entering in the fragility curves with the spectral displacement of the performance point, the structural damage probabilities are established and seismic risk scenarios can be then obtained.

Table 6.10. Parameters characterizing the fragility curves, for reinforced concrete buildings (RC) and unreinforced masonry buildings (M)

Building class	**Damage states thresholds**							
	$\overline{Sd}_1$ (cm)	β_1	$\overline{Sd}_2$ (cm)	β_2	$\overline{Sd}_3$ (cm)	β_3	$\overline{Sd}_4$ (cm)	β_4
Low-rise, RC	0.49	0.28	0.70	0.37	1.84	0.82	5.24	0.83
Mid-rise, RC	0.99	0.28	1.42	0.36	2.34	0.50	5.11	0.61
High-rise, RC	1.33	0.28	1.89	0.29	2.59	0.34	4.68	0.45
Low-rise, M	0.19	0.28	0.27	0.37	0.54	0.54	1.36	0.72
Mid-rise, M	0.44	0.40	0.63	0.50	1.20	0.75	2.91	0.70
High-rise, M	0.46	0.30	0.68	0.65	1.68	0.65	2.61	0.65

The response spectra for the deterministic and probabilistic hazard scenarios (Figure 6.7), together with the capacity curves described in Table 6.8, allowed obtaining the performance point and, using the corresponding fragility curves of Table 6.9 and the damage probability matrices of Table 6.10.

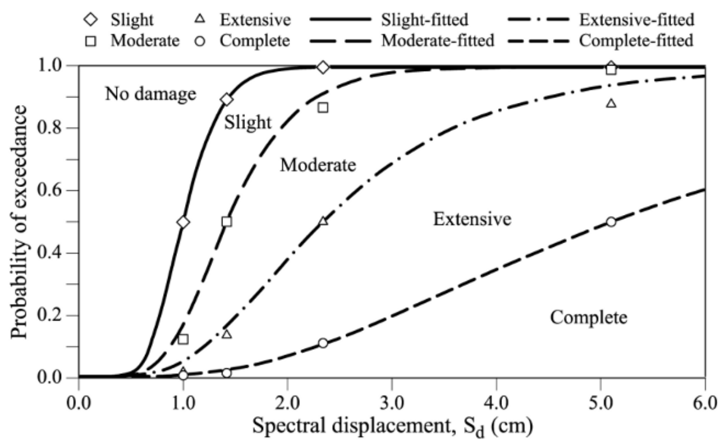

Fig. 6.8. Fragility curves for mid-rise reinforced concrete buildings

6.5. Final remarks

Both the vulnerability index and capacity spectrum methods provide excellent results, showing an excellent correlation with the main features of the built-up environment of Barcelona. It is clear in both cases that a city located in a low to moderate hazard region has paid no attention to the seismic performance of their buildings, and therefore, a high seismic vulnerability and a considerable risk are expected.

Another interesting feature of the described methodologies is their ability to draw the main characteristics of the built-up environment of the city, underlying the radial pattern of the damage. Downtown, where population density is higher and economy is more active, we find the highest vulnerability. The methods described here may be easily adapted to outline risk evaluations for other cities. Probably most of the vulnerability indices adopted for Barcelona may be slightly modified and directly used for obtaining risk scenarios for other cities of Spain and, in particular, for those situated in the Mediterranean region.

Acknowledgements

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PART II: VULNERABILITY ASSESSMENT

CHAPTER 7
VULNERABILITY ASSESSMENT OF HISTORICAL BUILDINGS

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7.1. Introduction

Going through the different ages of architecture history, it can be concluded that earthquakes always have, for many reasons, represented the main cause of damage and losses to the cultural heritage.

All ancient masonry buildings, including the biggest and most important monuments, have been constructed following the *rule of thumb*, learning from the experience of previous similar structures. Earthquake is a rare action and the builders experience was different from area to area and from time to time. In areas of high seismicity, where significant earthquakes occur quite often, buildings are characterized by constructive details and reinforcements specifically adopted to protect from seismic actions. In areas of moderate seismicity, these solutions may be found only in the buildings constructed immediately after a big earthquake, together with traditional repair techniques (tie rods, buttresses, scarp walls, foil arches between facing buildings); however, the awareness of the importance of these details disappears after two or three generations.

In the past, masonry buildings were constructed by refining the proportions of structural elements by a deep perception of the structural behaviour; this trial and error process took into consideration only static actions, mainly the dead loads. Notions such as dynamic amplification, damping, soil-structure interaction are not simple to be managed without a theoretical approach. For all these reasons, masonry buildings were proportioned to bear vertical loads and the static horizontal thrusts of arches and vaults. Considering the low tensile strength of masonry, both mortar joints placing and size and shape of structural elements were optimised. The seismic horizontal actions change significantly the thrust lines and usually produce widespread cracks and local collapses.

The vulnerability of historical buildings is thus related to the quality of masonry, the architectural shape and the dimensions and the presence of aseismic reinforcements.

Minor buildings in historical centres are usually very vulnerable due to the low quality of masonry material, the poor state of maintenance and the successive transformations (obstructions, raising up of buildings, partial demolitions).

Monumental buildings are equally vulnerable, even if for different reasons. Indeed, they are usually made by good quality materials, but their dimensions are significant: wide halls, thin long span vaults, slender towering or projecting parts, slender walls with large openings. Considering monuments, the importance and uniqueness of these structures may lead to dealing with the problem of seismic vulnerability through a detailed seismic analysis on each single building. However, even for the cultural heritage, seismic vulnerability is a problem that must be faced at territorial scale, due to the large number and the high density of monuments that are present in seismic prone areas. The aims of a vulnerability analysis of the monumental buildings in a big town or in a region are to be aware of the impact of an earthquake on the cultural heritage in the

C. S. Oliveira, A. Roca and X. Goula (eds.). Assessing and Managing Earthquake Risk, 135 - 158.

area, list the monuments by seismic vulnerability, plan preventive interventions for risk mitigation and manage the emergency after a big earthquake.

After recent Italian seismic events, the damage assessment to monumental buildings showed the high seismic vulnerability of this kind of structures and the relevance of this topic in a risk analysis both from the economic and the cultural point of view. In particular, churches turned out to be numerous and very vulnerable, compared to palaces or other ancient structures. In the case of low intensity earthquakes (Reggio Emilia, 1995; Piedmont, 2000 and 2003), churches have been the only types of structures that systematically suffered some damage. In the case of Umbria and The Marches earthquake (1997), besides the Assisi Basilica, more than 2000 churches suffered significant damage (Lagomarsino, 1998).

Even in the case of urban seismic risk scenarios, the number of monuments in a town is so large that a detailed evaluation on each single building is not possible. As an example, the RISK-UE Project, *An advanced approach to earthquake risk scenarios* (Mouroux et al., 2004), that aimed to develop seismic risk scenarios in some European towns, the monuments considered in the vulnerability assessment were: 67 in Barcelona (ESP), 194 in Catania (ITA), 151 in Nice (FRA) and 218 in Thessaloniki (GRE). According to the Ministry of Fine Arts, the list of monuments is huge, especially in the big cities: 3400 in Barcelona (ESP); 5000 in Bucharest (ROM); 700 in Sofia (BUL).

Well known methodologies for seismic risk scenarios of monumental buildings are not available, although some studies have been developed in the past, especially in Italy, on the observed vulnerability of a wide number of buildings (Doglioni et al., 1994; Lagomarsino and Podestà, 2004a; Lagomarsino and Podestà, 2004c). A vulnerability method has also been established for churches, through the statistical analysis of surveyed damage data (Lagomarsino and Podestà, 2004b). This chapter proposes a holistic approach to the seismic vulnerability assessment of monuments, which is able to profit from all the available information, both regarding the building vulnerability and the seismic hazard.

The seismic hazard map, in the town or in a larger area, may be defined by two different parameters:

- *Macroseismic intensity*: it is a hybrid measure of the seismic input, as it indirectly depends on the building vulnerability (even if the modern macroseismic scales try to overcome this restraint). The macroseismic intensity is useful when the hazard is derived by historical seismicity, both considering deterministic or probabilistic scenarios. Basically, intensity is a discrete variable, if you consider its definition in the macroseismic survey, but in a risk analysis it should be used as a continuous variable, if the vulnerability models are able to manage it properly.
- *Peak ground acceleration (PGA) and spectral values*: this is a mechanical representation of the seismic input, related to the structural response of an equivalent single degree of freedom system. PGA is a continuous variable, the spatial variability may be represented better than through the macroseismic intensity and, moreover, site effects may be taken into account, both as an amplification of the PGA and changing the spectral shape.

The vulnerability of a monumental building is its intrinsic predisposition to be affected and suffer damage as a result of the occurrence of an event of a given severity. It is represented by a model able to assess the physical damage (in probabilistic terms) as a function of the intensity or the PGA/spectrum.

In the case of a macroseismic scenario, a vulnerability curve correlates the intensity to a histogram of damage grades, D_k (k=0,1,2,3,4,5), expressed by the mean damage grade μ_D (a continuous parameter, $0<\mu_D<5$), and a proper discrete probabilistic distribution. This *macroseismic approach* is based on the observed vulnerability, as these curves have been obtained, for classes of buildings, through damage assessment data, collected after earthquakes of different intensities. The vulnerability curve is defined by two parameters, the vulnerability index V and a toughness coefficient β, which should be evaluated from the information about the building.

In the case of a PGA scenario, the vulnerability model is the capacity spectrum method, in which the capacity curve represents the non-linear behaviour of the building to seismic horizontal actions, through an equivalent single degree of freedom system (Freeman, 1998; Fajfar, 2000). On the capacity curve, some damage thresholds d_k (k=1,2,3,4) may be identified. Once the performance point is obtained, through the intersection of the capacity curve with a properly reduced spectrum, the probability of occurrence of each damage state is obtained by means of fragility curves. This method constitutes a *mechanical approach*, as a capacity curve may be obtained through more or less detailed mechanical models, but a validation with observed vulnerability data is necessary, due to the complexity of modelling historical masonry structures.

Both vulnerability models depend on the characteristics of the building (typology, materials, dimensions and shape, constructive details) and their assessment can be more or less accurate according to the level of knowledge of the exposed elements. Thus, a three levels approach is proposed (Level 0, 1 and 2), in relation to the accuracy and the meaningfulness of the collected data, both for the macroseismic and the mechanical approach. This allows using for the risk analysis on the cultural heritage in a given region, without distinction, the most suitable method depending on the characteristics of the hazard scenario; moreover, the same model may be applied to all the monuments, allowing a unique comparable damage scenario, but the evaluation results are more or less accurate in relation to the level adopted (usually level 1 or 2 for the most important buildings and level 0 for the others).

Level 0. The lowest level of information is the inventory of monuments, without any specific data except the typology (church, monastery, palace, villa, tower, etc). Even in this case a very rough evaluation can be made with the macroseismic or the mechanical approaches. In the first case a vulnerability index is assigned to each typology (some values are proposed in 7.3, based on observed vulnerability and expert judgement); if a mechanical approach has to be used, a capacity curve may be defined, in order to get comparable results with the two approaches. The vulnerability assessment at this level may be useful for an overall knowledge of the seismic risk of the cultural heritage in a big town or in a region, in order to plan preventive interventions for the risk mitigation, establishing priorities and allocating funding (Bianchi and Accardo, 1998).

Level 1. In this case some data are available, in particular the one connected to the structural seismic behaviour: regularity (in plan and elevation), quality of materials, dimensions (number of floors, slenderness of elements), state of maintenance, transformations and interventions. Usually a quick field survey may provide this kind of information, when it is not contained in the available databases. In the macroseismic approach, to each one of these parameters a score is awarded, which modifies the vulnerability index assigned to the typology. In this way, each monument is characterized by its own vulnerability index, and the vulnerability assessment is more accurate and allows making a list of vulnerabilities for the monuments of the same type. The capacity curves may be derived as for level 0, not using a mechanical model but directly from the vulnerability index and taking into consideration some structural parameter (e.g.: dimensions and period of vibration).

Level 2. Going deep in the damage observation, recurrent damage and collapse mechanisms have been observed, which usually don't involve the whole structure but only single architectonic parts, named macroelements, which are characterised by a mostly autonomous structural behaviour in comparison with that of the rest of the building; for example, in the case of churches, façade, nave, triumphal arch, dome, apse, bell tower are the most important macroelements (Doglioni et al., 1994). At this level, the vulnerability assessment methodology analyses the seismic behaviour in terms of local mechanisms, but it is also possible to define a global vulnerability of the structure by a proper combination of the contributions of different macroelements. Due to the wide experience with churches, in this chapter reference is made to this typology but the method may be extended also to other types of monumental structures. The macroseismic approach is again based on a vulnerability index that is obtained in this case through scores related to a detailed survey in the macroelements of the church, to be performed by means of a proper form (Lagomarsino et al., 2004a). The diagnostic survey of the possible damage and collapse mechanisms in each single part of the church, allows us to take into account the source of vulnerability information and the effective constructive details of each macroelement. The mechanical approach may consider simplified mechanical models, representative of the most probable local collapse mechanisms. As local mechanisms are considered, only a sub-structure has to be modelled (macroelement); the application of the safe and unsafe theorems of limit analysis, applied to masonry structures (Heyman, 1966; Heyman, 1982), may be very useful to this aim. The masonry structure is considered to be made of rigid blocks, due to the low tensile strength and the relevant compressive strength and stiffness. With this approach, only few geometrical parameters and technological details are needed, which can be obtained during a field survey. An incremental non-linear limit analysis may be performed, in order to evaluate the displacement capacity of the masonry structure, by an approximate approach that makes use of the capacity spectrum method (Lagomarsino et al., 2004b).

Finally, it is worth noticing that all the above mentioned methods are far from the close examination that is needed in the design of seismic improvement interventions on monumental structures (Level 3). The high complexity that is necessary for an exhaustive study is outside the scope of a vulnerability analysis, because it is impossible to be implemented for a territorial analysis.

7.2. The observed vulnerability in historical buildings

Damage assessment represents a remarkable source of information of the seismic emergency, as an earthquake represents a test of the actual vulnerability of structures. This activity, usually coordinated by the Civil Protection Department and made using proper survey forms, in order to get systematic and objective data, may be useful to: 1) verify if the structure is fit to use in the emergency (green, yellow or red tag); 2) decide for provisional interventions (propping, hopping), in order to prevent further damage due to after shocks; 3) evaluate economic losses, to be charged to insurance companies or for funding from the State; 4) make a preliminary diagnosis of the building performance, in order to design the proper repairing and strengthening interventions.

Besides the above mentioned uses for the single building and the emergency management, the damage assessment gives a lot of data useful for statistical analyses aimed to develop observational vulnerability models, related to homogeneous typologies, which can also consider the influence of some details and the effectiveness or the deficiencies of the most used strengthening interventions.

7.2.1. DAMAGE AND VULNERABILITY ASSESSMENT OF CHURCHES

During the last twenty years, the damage to churches caused by various earthquakes in Italy has been systematically assessed and interpreted from the structural point of view. The seismic response of churches showed recurrent behaviours, according to damage and collapse mechanisms of the different architectonic parts, called macroelements, which behave almost independently. Typical examples of macroelement are the façade, the nave, the triumphal arch, the dome, the apse and the bell tower.

In particular, the damage to the churches after the Umbria and Marche earthquake, in 1997, has been assessed by a proper form, which considers eighteen indicators, each one representative of a possible collapse mechanism in a macroelement:

- façade: 1) façade overturning; 2) overturning of the gable; 3) in plane mechanisms;
- nave and transept: 4) transversal vibration of nave or of the transept; 5) longitudinal vibration of the central nave; 6) vaults of the central nave; 7) vaults of the lateral naves and of the transept;
- triumphal arch: 8) kinematics of triumphal arches;
- dome and “tiburio”: 9) collapse of the dome and the “tiburio”;
- apse: 10) overturning of the apse; 11) vaults of the apse and of the presbytery;
- other mechanisms: 12) overturning of walls (transept façade, chapels); 13) shear failure of the walls; 14) hammering and damage in the roof covering; 15) interaction with adjacent structures;
- bell tower: 16) global collapse of the bell tower; 17) mechanisms in the bell cell;
- bell gable, spires and projections: 18) overturning of standing out elements.

For each mechanism, the survey form asks for the suffered damage level (explained by proper sketches - Figure 7.1), and some constructive characteristics and reinforcements of the macroelements, relevant to judging the seismic vulnerability (Lagomarsino, 1998). This is a diagnostic approach, as it is not aimed to measure the length and the size of cracks, but to recognize the mechanisms started up by the earthquake and their severity and dangerousness with respect to the local collapse. A damage index and a vulnerability index may be defined by the data in the form.

2. DAMAGE AT THE TOP OF THE FACADE	□
CRACKS IN THE TOP PART OF THE FACADE	□□□
□ Facade weakened by wide openings □ Lack of a connection with the roof covering	

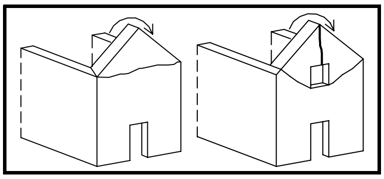

Fig. 7.1. Abstract of the damage assessment church form: survey of the damage in a macroelement in terms of collapse mechanisms and vulnerability indicators

More than 2000 churches were assessed in Umbria and Marche, characterized by different typologies (from basilicas to small rural churches) and struck by different levels of macroseismic intensity I (from V to VIII of the MCS scale, according to the survey of INGV, the Italian National Institute of Geophysics and Volcanology).

The statistical analysis of the data produced a Damage Probability Matrix (DPM) for the churches, that is a matrix in which for four different values of the macroseismic intensity, the probability histogram of the damage levels is listed. The five damage levels are defined according to the modern macroseismic scales, in particular the EMS98, *European Macroseismic Scale* (Grünthal 1998): 0) no damage; 1) slight damage; 2) moderate damage; 3) heavy damage; 4) very heavy damage; 5) destruction. For each intensity, the mean damage grade μ_D may be defined, by the probability P_k of each damage grade D_k:

$$\mu_D = \sum_{k=1}^{5} kP_k \tag{7.1}$$

The damage histograms (Figure 7.2) are well fitted by the binomial distribution, which is defined only by one parameter, just the mean damage grade μ_D:

$$P_k = \frac{5!}{k!(5-k)!}\left(0.2\mu_D\right)^k \ (1-0.2\mu_D)^{5-k} \tag{7.2}$$

Figure 7.2 shows the gradual increase of the damage with the intensity; this trend appears very regular if you consider the mean damage grade μ_D. Many churches have been surveyed with this approach after earthquakes in other regions of Italy; the statistical analyses corroborated the correlation obtained in Umbria and Marche.

The DPM considers all the churches as a homogeneous typology, characterizing on average its vulnerability. However, the survey form allows taking into account the specific vulnerability of each church, evaluated through a vulnerability index. Dividing the sample of the churches, in areas that suffered the same seismic intensity, into smaller samples characterized by ranges of the vulnerability index V, a more refined correlation may be set up (Lagomarsino and Podestà, 2004b), which also considers this intrinsic parameter of the church: Figure 7.2 shows the gradual increase of the damage with the intensity; this trend appears very regular if you consider the mean damage grade μ_D. Many churches have been surveyed with this approach after earthquakes in other regions of Italy; the statistical analyses corroborated the correlation obtained in Umbria and Marche.

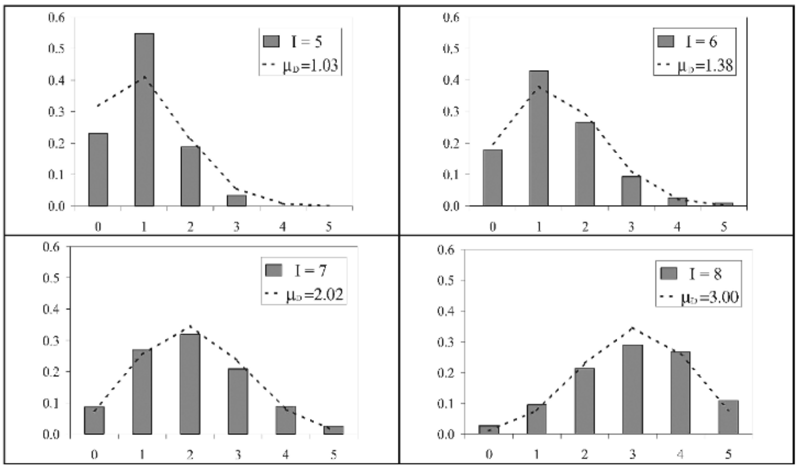

Fig.7.2. Damage probability histograms for the churches

The DPM considers all the churches as a homogeneous typology, characterizing on average its vulnerability. However, the survey form allows taking into account the specific vulnerability of each church, evaluated through a vulnerability index. Dividing the sample of the churches, in areas that suffered the same seismic intensity, into smaller samples characterized by ranges of the vulnerability index V, a more refined correlation may be set up (Lagomarsino and Podestà, 2004b), which also considers this intrinsic parameter of the church:

$$\mu_D = 2.5\left[1+\tanh\left(\frac{I+6.25V-13.1}{3}\right)\right] \quad (7.3)$$

where V assumes values between 0.67 and 1.22 (for the more vulnerable churches).

7.2.2. VULNERABILITY CURVES OF PALACES AND CHURCHES

The vulnerability curve (7.3) represents a model to evaluate the probability damage distribution of a church, for which the vulnerability index V has been evaluated by a proper survey, as a function of the hazard, in terms of the macroseismic intensity I. A similar model has been derived for current buildings (Giovinazzi and Lagomarsino, 2004), on the basis of the European Macroseismic Scale, using fuzzy set theory, and after a validation through damage observation data.

The method is based on the assignment to any building, or to a group of buildings, of a vulnerability index V, which is obtained as the sum of the typological vulnerability index V_0, related to the EMS98 building classification, and the vulnerability scores, assigned to some relevant parameters of the construction (state of maintenance, material quality, structural regularity, etc.). The mean damage grade is given by:

$$\mu_D = 2.5\left[1+\tanh\left(\frac{I+6.25V-13.1}{2.3}\right)\right] \qquad (7.4)$$

The vulnerability index V usually varies from 0 (for structures with a high level of earthquake resistance design) to 1 (for the very vulnerable poor masonry buildings).

Among the different typologies there is the one of massive stone buildings. Monumental palaces may be, on average, associated to this typology, because their construction is typically characterized by good quality materials and craftsmanship. For massive stone buildings V_0=0.62, and the vulnerability index may vary, according to the vulnerability scores, from 0.3 to 0.86 (a plausible range is $0.49<V<0.79$).

It is worth noting that the vulnerability curves of churches (7.3) and palaces (7.4) are similar, with the exception of denominator, which controls the rate of increase of the damage with the intensity. It is defined as the ductility index, Q. The vulnerability curves of the churches and of the monumental palaces are compared in Figure 7.3. Due to higher values, on average, of the vulnerability index, churches turn out to be more vulnerable for the lower intensities; actually, in the case of minor earthquakes in Italy, the churches always exhibited a higher damage among the built environment. The higher ductility of the churches (Q=3 for churches, Q=2.3 for palaces) determines that for higher intensities, the seismic response tends to be similar to the one of palaces.

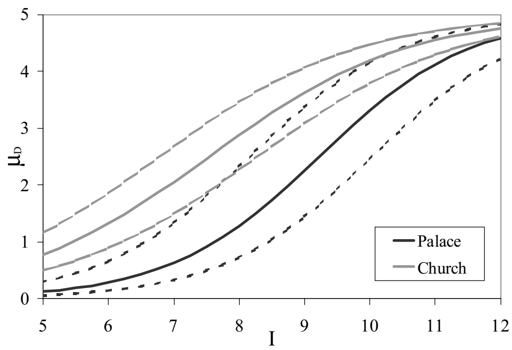

Fig. 7.3. Vulnerability curves of palaces and churches (mean value, plausible range)

7.3. The vulnerability assessment methodology

Also in the case of historical buildings, a vulnerability model suitable for application at the territorial scale has to be referred to a typological classification. Considering the wide variety of the artefacts that constitute the cultural heritage in the world, according to geographic location, architectural styles and ages of construction, this classification is not a straightforward task. However, for the sake of a simplified structural evaluation of the seismic vulnerability on a wide population of monuments, a typological classification is usually possible, gathering into groups the structures which are similar with reference to the architecture and the seismic behaviour.

An example of typological classification, which may be considered as a reference for monuments in European countries, is the following: palace, church, monastery/convent, mosque, tower, obelisk, theatre, castle, triumphal arch, arch bridge.

Considering the two vulnerability approaches described in the introduction of this chapter (macroseismic and mechanical), in the following paragraphs the basis of the methods are stated, both for level 0 and level 1 grades of detail in the analysis.

7.3.1. BASIS OF THE MACROSEISMIC APPROACH (LEVEL 0)

The damage assessment after earthquakes and the definition of the macroseismic scale (EMS98) allow us to state the observational vulnerability model, through a correlation between the intensity I of the earthquake and the mean damage grade μ_D, which represents the mean value of the probability histogram of the damage grades D_k (k=0,1,2,3,4,5), typical of easily observable levels of damage, in terms of cracks and deformations. The vulnerability curves are:

$$\mu_D = 2.5\left[1 + \tanh\left(\frac{I + 6.25V - 13.1}{Q}\right)\right] \qquad (7.5)$$

The model is defined by two parameters, the vulnerability index V and the ductility index Q. The vulnerability index V ranges from 0 to 1, in the case of the six building types defined in EMS98; for masonry building, V is greater than 0.4. In the case of churches, V can assume values between 0.67 and 1.22. A decrease of V equal to 0.16 means that you need an increase of one degree in the intensity of the earthquake, in order to produce the same damage grade. The ductility index Q determines the rate of increase in the damage with intensity. If Q=2.3 (as for buildings), one intensity degree corresponds to one damage grade; greater values of Q are typical of ductile structures.

Reference values for the other monumental types may be deduced from them, according to expert judgment and taking advantage of some observed data on each typology. The values in Table 7.1 may be used for a Level 0 vulnerability assessment, having at disposal only the list of monuments in the city or in the region.

Table 7.1. Parameters for the macroseismic and the mechanical models

Model parameters	Macroseismic		Mechanical		
Typology	V_0	Q	T (s)	a_y (g)	μ
Palace	0.62	2.3	0.35	0.35	4.8
Church	0.89	3.0	0.40	0.09	7.5
Monastery/Convent	0.74	2.3	0.40	0.23	4.3
Mosque	0.81	2.6	0.35	0.15	6.1
Tower	0.78	2.0	0.70	0.13	3.4
Obelisk	0.74	3.0	1.00	0.06	7.5
Theatre	0.70	2.3	0.45	0.23	4.3
Castle	0.54	2.0	0.25	0.56	4.8
Triumphal arch	0.58	2.6	0.60	0.23	5.5
Arch bridge	0.46	2.3	0.30	0.63	5.4

Once the hazard scenario is known, damage level can be evaluated immediately for each structure (damage scenarios) and obtain a list of the monuments by their risk can be obtained. The mean damage grade μ_D, given by (7.5), represents a synthetic meaningful parameter for the damage scenario; Figure 7.4 shows the mean vulnerability curves for the different monumental typologies. If a probabilistic evaluation is needed, the probability P_k (k=0,1,2,3,4,5) joined to each damage grade is given by the binomial distribution (7.2); these values may be useful for more detailed scenarios, aimed at showing, for example, the probability of collapse of each building (P_5) or the probability that the building is unfit for use in the emergency ($P_3+P_4+P_5$). The fragility curves are then:

$$P\left[D_k \middle| \mu_D\right] = \sum_{i=k}^{5} P_i = \sum_{i=k}^{5} \frac{5!}{i!(5-i)!} \left(0.2\mu_D\right)^i (1-0.2\mu_D)^{5-i} \qquad (7.6)$$

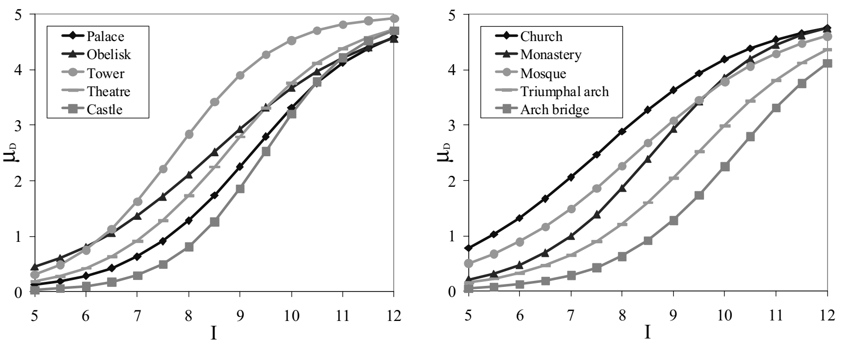

Fig. 7.4. Mean vulnerability curves for the different types of monumental buildings

7.3.1.1. *The macroseismic vulnerability assessment (Level 1)*

It is evident that a vulnerability index assigned to a monument simply by a typological classification represents an average value, which doesn't take into account the distinctiveness of the single building and doesn't allow us to single out the most vulnerable structures among buildings of the same type. To refine the vulnerability assessment a quick survey is at least necessary, with the purpose of collecting by proper survey forms some relevant parameters, such as: state of maintenance, quality of materials, structural regularity (in plan and in elevation), size and slenderness of relevant structural elements, interaction with adjacent structures, presence of retrofitting interventions, site morphology.

Vulnerability scores V_k may be awarded to each one of the above mentioned parameters and the vulnerability index of each monument may be refined, modifying the typological value:

$$V = V_0 + \sum V_k \qquad (7.7)$$

where the summation is extended to all the available modifiers. The vulnerability scores may assume different values for different typologies; moreover, some relevant information may be distinctive only of one typology (e.g.: the presence of a raising façade in churches).

The choice of the specific vulnerability parameters has been made empirically, on the basis of the observation of the typical damage. This approach is similar to many well-known vulnerability procedures, proposed in different countries for ordinary buildings (Benedetti and Petrini, 1984; ATC-21, 1988). The values of the modifying scores have been tuned, for churches and palaces, by statistical analysis of data that come out from damage assessment (§ 7.2.1). Some reference values are proposed in Table 7.2.

Table 7.2. Reference values for vulnerability scores Vk of the main parameters

Parameter	V_k
state of maintenance	very bad (0.08) – bad (0.04) – medium (0) – good (-0.04)
quality of materials	bad (0.04) – medium (0) – good (-0.04)
planimetric regularity	irregular (0.04) – regular (0) – symmetrical (-0.04)
regularity in elevation	irregular (0.02) – regular (-0.02)
interactions (aggregate)	corner position (0.04) – isolated (0) – included (-0.04)
retrofitting interventions	effective interventions (-0.08)
site morphology	ridge (0.08) – slope (0.04) – flat (0)

7.3.2. STATEMENT OF THE MECHANICAL APPROACH (LEVELS 0 AND 1)

The use of a mechanical approach for the vulnerability assessment has been proposed, in particular for current buildings, by HAZUS (NIBS, 1997, 1999 and 2002), which considers the capacity spectrum method. This method permits evaluating the expected seismic performance of a structure, assumed as an equivalent non -linear Single Degree of Freedom system, by intersecting, in spectral coordinates (S_d, S_a), its seismic capacity curve with the seismic demand, described by the Acceleration-Displacement Response Spectra (ADRS), adequately reduced in order to take into account the inelastic behaviour.

7.3.2.1. *Application of the capacity spectrum method for the vulnerability assessment*
The capacity curve of a structure should be obtained by a pushover analysis, but in the case of a territorial vulnerability assessment, at levels 0 or 1 of available data, a bilinear capacity curve must be defined by the typology taking into account some qualitative information. In the case of monuments, the characteristics of masonry structures suggest neglecting the possibility of a global hardening behaviour; thus, the capacity curve is defined by three parameters: the fundamental period (T), the yielding spectral acceleration (a_y) and the ductility (μ).

Instead of using overdamped spectra, for which we need to know the equivalent damping as a function of the displacement and to evaluate the performance point by an iterative procedure (Freeman 1998), in the case of bilinear capacity curves it is preferable to adopt the inelastic spectra approach (Fajfar, 2000), which can be applied in a very simple way and is reliable for structures characterized by high hysteretic dissipation.

A mechanical approach is used when the hazard scenario gives the elastic response spectrum (at 5% damping) $S_{ae}(T)$, usually by discrete values for fixed periods or by the peak ground acceleration a_g and a predefined spectral shape (related to the local soil conditions). In both cases a characteristic period T_C may be defined, which separates the periods of almost constant spectral acceleration ($T<T_C$) by the almost constant spectral velocity range ($T>T_C$). The performance point, which is the spectral displacement demand (target displacement), is obtained by:

$$S_{d*} = \begin{cases} [1+(q-1)T_C/T]d_y & T < T_C \text{ and } q > 1 \\ qd_y & T_C \le T < T_D \text{ or } q \le 1 \\ S_{ae}(T_D)T_D^2/4\pi^2 & T \ge T_D \end{cases} \quad (7.8)$$

where: T is the period of the structure; $d_y=a_yT^2/(4\pi^2)$ is the yielding displacement of the structure, $q=S_{ae}(T)/a_y$ is the ratio between the demand to an elastic system and the strength of the non -linear structure; T_D is the period that defines the constant spectral displacement range.

The capacity curve represents the progress of the structural response to horizontal seismic actions, from the initial undamaged elastic behaviour, through the formation and the development of cracks, to the loss of stability, near to collapse. Four damage states, which are usually related to performance levels of the structure, may be defined according to Table 7.3, where the mean values $S_{d,k}$ (k=1,2,3,4) of the displacement thresholds are indicated. It is worth noting that the bilinear behaviour is an approximation of the actual curved response, usually made considering an equivalent period (of the cracked structure) and with an equivalent energy dissipation; in particular, the slight damage occurs before yielding, while moderate damage, corresponding to the achievement of the maximum strength, is attained for a spectral displacement greater than d_y.

Table 7.3. Mean values of the damage state thresholds $S_{d,k}$

D_k	damage state	performance level	$S_{d,k}$
1	Slight	fully operational	$0.7d_y$
2	Moderate	Operational	$1.5d_y$
3	Heavy	life safe	$0.5(1+\mu)d_y$
4	Complete	near collapse	μd_y

Figures 7.5 show the capacity curves of two different structures (rigid and flexible), with the damage state thresholds and the target displacement, obtained by (7.8) in the case of two different demand spectra.

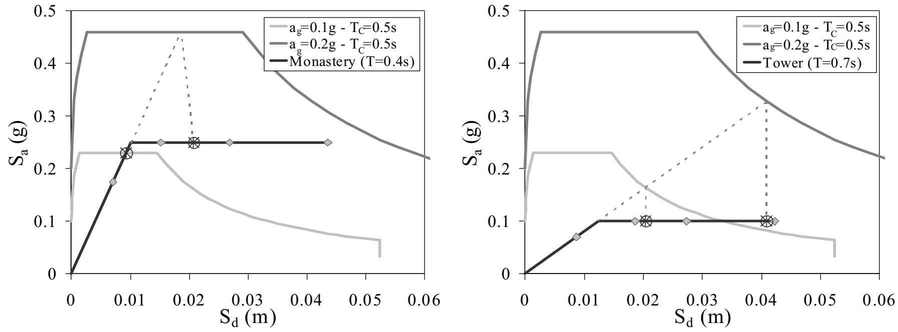

Figure 7.5. Capacity curves, damage state thresholds and evaluation of the target displacement for rigid ($T<T_C$) and flexible structures ($T>T_C$), for two demand spectra

As for the macroseismic approach, the output of the vulnerability assessment is the probability distribution of the expected damage state. The fragility curves give the probability that the damage is equal to or higher than a certain state D_k, as a function of the target displacement S_{d*} (performance point, obtained by eqn 7.8); a lognormal cumulative probability function may be used:

$$P\left[D_k \middle| S_{d*}\right] = \Phi\left[\frac{1}{\beta_k}\ln\left(\frac{S_{d*}}{S_{d,k}}\right)\right] \tag{7.9}$$

where Φ is the normal cumulative distribution function and β_k is the normalized standard deviation of the natural logarithm of the displacement threshold $S_{d,k}$. The probability histogram of the damage state is then given by:

$$P_4 = P\left[D_4 \middle| S_{d*}\right] \quad P_k = P\left[D_k \middle| S_{d*}\right] - P_{k+1} \quad (k = 1,2,3) \quad P_0 = 1 - P_1 \tag{7.10}$$

Comparing the damage state in Table 7.3 with the damage grades of the macroseismic approach (see § 7.2.1), a direct correspondence of the first four levels can be observed, while the fifth damage grade (destruction) cannot be defined by a mechanical approach, as the softening behaviour should be included in the capacity curve.

As the distribution of the macroseismic method has been validated by observed vulnerability data, the binomial distribution may be used to take out from the probability P_4, the part that corresponds to the building collapse (P_5). It results that a reliable estimation is given by:

$$P_5 = 0.09\,\mathrm{senh}(0.6\mu_{DS})P_4 \qquad \text{where } \mu_{DS} = \sum_{k=1}^{4} kP_k \tag{7.11}$$

Moreover, the use of binomial distribution allows us to define a reliable estimation of the coefficient β_k, which results are dependent on the ductility μ of the capacity curve, while the same value can be assumed for all the damage state:

$$\beta_k = 0.4\ln\mu \quad (k = 1,2,3,4) \tag{7.12}$$

Fragility curves are shown in Figure 7.6a, with reference to the more flexible structure in Figure 7.5 (tower). Figure 7.6b shows the histograms of damage state probability, for the two different values of the peak ground acceleration; light grey represents the share of P4 that is assumed as destruction (D5), according to (7.11).

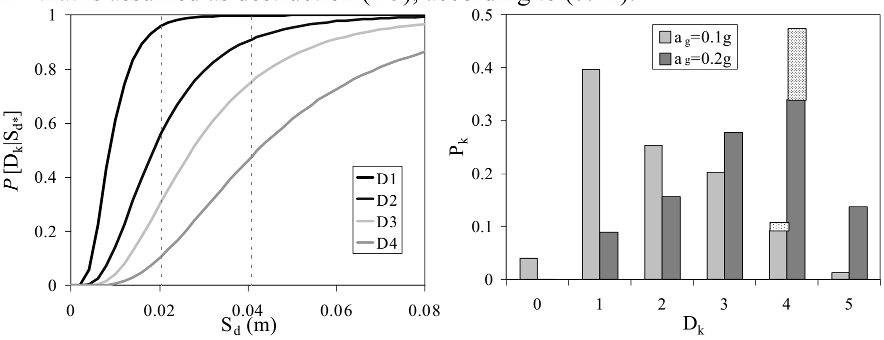

Figure 7.6. Evaluation of damage states probability for the two structures of Figure 7.5: a) fragility curves; b) damage histograms

If the hazard scenario is defined only by the peak ground acceleration a_g, fixing the spectral shapes for one or a limited number of soil conditions, the fragility curves (7.9) may be expressed directly as a function of a_g:

$$P\left[D_k \middle| a_g\right] = \Phi\left[\frac{1}{\beta_k}\ln\left(\frac{a_g}{a_{g,k}}\right)\right] \tag{7.13}$$

where $a_{g,k}$ is the ground acceleration that produces the damage state D_k (k=1,2,3,4), which is obtained, for each structural typology, by substituting the value $S_{d,k}$ of Table 7.3 in eq. (7.8).

7.3.2.2. *Definition of capacity curves for monumental buildings*

If the vulnerability assessment is performed with information at level 0 or 1, it is impossible to evaluate capacity curves even by a simplified structural analysis, because the data are too poor. However, it is possible to deduce capacity curves for the monumental typologies in the classification of Table 7.1, to be used for a very rough vulnerability assessment. This can be made by establishing an equivalence of the results that are obtained by the macroseismic and the mechanical approach.

To this aim, first of all it is necessary to set a correlation between intensity I and peak ground acceleration a_g. These parameters are completely different, the second being a physical parameter of the motion, variable from point to point due to local soil conditions, and the former a subjective measure, average in a wide area, that implicitly includes the vulnerability itself (even if the EMS98 scale tries to overcome this limitation). Different correlations have been proposed in the literature and they are extremely scattered, but most of the analytical relations may be expressed in this form:

$$a_g = c_1 c_2^{(I-5)} \qquad I = 5 + \frac{1}{\ln c_2}\left(\ln a_g - \ln c_1\right) \tag{7.14}$$

Moreover, the spectral shape has been assumed to have a constant value equal to $s a_g$, for $T<T_C$, followed by a constant velocity phase ($S_a(T)=2.5a_gT_C/T$); s and T_C are free parameters that define the demand spectrum.

One of the three parameters of the mechanical method (T, a_y, μ) have to be assumed on the base of the typology, as the macroseismic method is defined only by two parameters (V, Q); the period T has been chosen, the easiest one to be estimated by expert judgement and the one that has the lowest influence on the evaluation of the displacement demand. By requiring that the two approaches give the same results for damage levels 1 and 4, we have the necessary relations for deriving the parameters of the mechanical method:

$$\begin{aligned}
T < T_C \quad & \begin{cases} a_y = 1.43 s c_1 c_2^{(8.1-6.25V-0.95Q)} \\ \mu = 1 - \dfrac{T_C}{T} + 0.7\dfrac{T_C}{T} c_2^{1.35Q} \end{cases} \\
T \geq T_C \quad & \begin{cases} a_y = 1.43 s c_1 c_2^{(8.1-6.25V-0.95Q)} \dfrac{T_C}{T} \\ \mu = 0.7 c_2^{1.35Q} \end{cases}
\end{aligned} \tag{7.15}$$

As it is evident, having fixed the period T, the yielding spectral acceleration a_y mainly depends on the vulnerability index V, while the ductility μ is correlated only with the ductility index Q, which influences the rate of damage increasing with the intensity. Comparing the results of the two methods with reference to damage levels 2 and 3, it comes out, on average, to be a very good agreement; this confirms that the values of $S_{d,k}$ adopted in Table 7.3 are reliable, due to the validation with the observational approach.

In Table 7.1 values are proposed for the monumental types; the corresponding capacity curves are shown in Figure 7.7. The values of parameters a_y and μ have been obtained by adopting for the spectral shape: s=2.5, T_C=0.4 s, and for the correlation I-a_g: c_1=0.03g, c_2=1.8. Both yielding accelerations and ductilities seem to be realistic, even if derived analytically; this represent a cross-validation of the macroseismic method and of the vulnerability and ductility indexes assumed.

The mechanical method here proposed may be applied also if the vulnerability assessment is made at level 1. In this case, the available information allows us to refine the typological vulnerability index, by means of the vulnerability scores (eq. 7.7, Table 7.2). For each single monument, a proper fundamental period may be assumed, considering the size and other characteristics of the structure, and a capacity curve may be defined by (7.15), using a I-a_g correlation (7.14) calibrated in the study area.

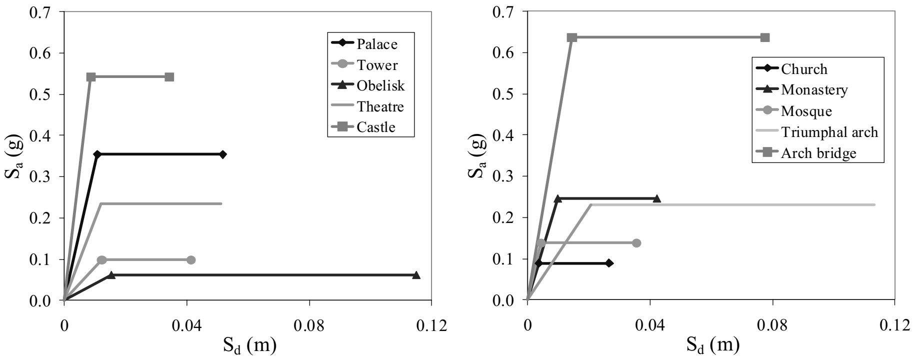

Fig. 7.7. Capacity curves for monumental buildings

7.4. Macroseismic vulnerability assessment of churches (Level 2)

The limitation of level 1 methodology is that vulnerability is considered in a global way. On the contrary, the damage observation has highlighted how, according to the architectonic complexity of monumental buildings (geometry, constructive phases, transformations, etc.) and to the poor tensile strength of the masonry, the damage and collapse often takes place locally. Thus, a proper approach should be the evaluation of the vulnerability in each macroelement, an architectonic element of the building characterized by a substantially independent seismic response. A comprehensive vulnerability index, representative of the overall behaviour of the monument, may be defined as a weighted average of the characteristics in each macroelement.

As a result of the wide experience in assessing damage and vulnerability to churches after the earthquakes in Italy (see § 7.2.1), a level 2 macroseismic vulnerability method has been stated (Lagomarsino et al., 2004a), which considers 28 damage mechanisms, able to represent the seismic behaviour of churches of different architectonic styles and configurations (one or three naves, with or without transept, etc.).

The vulnerability of each macroelement is the result of two complementary shares: 1) the vulnerability indicators (poor masonry, slenderness of elements, presence of thrusting elements, etc.), weakness of the macroelement with reference to a specific collapse mechanism; 2) the specific aseismic reinforcements (tie rods, buttresses, etc.), constructive details that reduce the vulnerability.

The use of this methodology in different regions in Italy and also in some other countries has proved its wide applicability and effectiveness. The vulnerability index which is obtained is fully compatible with the one defined in § 7.3.1; thus, for the vulnerability assessment of churches in a wide region, it is possible to assess the vulnerability of the most important churches with the level 2 vulnerability form, and evaluate the others with a quick survey (level 1) or only with available data (level 0).

7.5. A mechanical model for capacity spectrum method on monuments (Level 2)

The mechanical approach at level 0 and 1 is based on capacity curves directly derived from the macroseismic vulnerability index, in order to have a methodology applicable with a hazard scenario in terms of peak ground acceleration and spectral values. If information is detailed (level 2), the capacity curve should be defined by a simplified mechanical model, for the sake of a more reliable and consistent approach.

However, only in the case of some monumental typologies (tower, obelisk, etc.) is the definition of a capacity curve describing the global behaviour of the monument conceptually correct. Usually, as shown in the previous paragraph, the damage and collapse mechanisms are related to parts of the building (macroelements) and the capacity curve should represent this local behaviour. Thus, the first step of a vulnerability assessment with a mechanical approach is to single out the weakest macroelement and the corresponding collapse mechanism.

In order to define the capacity curve, a pushover analysis by the non-linear finite element method (f.e.m.) should be done. However, modelling a complex historical structure is not a straightforward task (lack of homogeneity in the materials, uncertainty in the connection between elements, etc.) and reliable constitutive non-linear models for masonry are not widespread (Lourenço et al., 1997; Calderini and Lagomarsino, 2004).

An alternative approach for a pushover analysis on a historical masonry structure (Lagomarsino et al., 2004b) is the use of the kinematic theorem of limit analysis (Heyman, 1966), based on the a priori selection of the collapse mechanism, that is the transformation of the structure in a kinematic mechanism, by positioning a sufficient number of hinges or sliding planes. Each resulting block is subjected to dead loads and to horizontal seismic action, proportional to the dead loads through a coefficient α. Under the hypothesis of non-tensile strength of masonry, unlimited compressive strength and rigid blocks, the seismic coefficient α_o that induces the loss of equilibrium is obtained by the principle of virtual works:

$$\alpha_0\left(\sum_{i=1}^{n} W_i\delta_{x,i} + \sum_{j=n+1}^{n+m} W_j\delta_{x,j}\right) - \sum_{i=1}^{n} W_i\delta_{y,i} + \sum_{h=1}^{o} F_h\delta_h = L_{fi} \qquad (7.16)$$

where:

n is the number of carried weights W_i, applied in their centres of gravity to the blocks;

m is the number of weights W_j, which are not born by the macroelement but that induce a horizontal seismic action, as not effectively connected to other building elements;

o is the number of external actions F_h, such as the thrusts of roof, arches and vaults;

$\delta_{x,i}$ and $\delta_{x,j}$ are the virtual horizontal displacements of the points where the weights W_i and W_j are applied (positive if coherent with the mechanism development);

$\delta_{y,i}$ is the virtual vertical displacement of the point where the weight W_i is applied (positive if upward);

δ_h is the virtual displacement of the point where F_h is applied, in this direction;

L_{fi} is the virtual work of possible internal actions (friction sliding, interlocking, etc.).

The virtual displacements are obtained by applying to the kinematic mechanism an infinitesimal deformation; for example, if an infinitesimal rotation θ_k is applied to the block k, the rotations of the other blocks are obtained by the kinematic mechanism, only considering geometry, and the same is for the displacements of each relevant point.

A pushover analysis is an analysis of the seismic performance of the structure, not only by the evaluation of coefficient α till maximum strength is reached but also by increasing the displacements till collapse. In this case, as an earthquake is a dynamic action, the static loss of equilibrium doesn't correspond to the collapse, and the kinematic mechanism is able to sustain some horizontal action also after its activation. To this end, an incremental kinematic analysis may be performed, by applying (7.16) to different configurations, obtained incremental displacement d_k of a properly chosen control point k. As finite displacements are considered, the seismic coefficient α usually decreases gradually because of the reduction of the stabilizing contribution of the dead loads; an exception is in the case of presence of forces F_h that increase their value with the progression of the kinematic mechanism (tie rods). The incremental analysis must be performed till the zeroing of the coefficient α, that occurs for a displacement $d_{k,0}$; if the different actions may be considered as constant during the progression of the kinematic mechanism, the pushover curve is well approximated by a straight line:

$$\alpha = \alpha_0\left(1 - d_k / d_{k,0}\right) \qquad (7.17)$$

As an example, the typical collapse mechanism of a triumphal arch in a church is shown in Figure 7.8a. The progress of the coefficient α with the displacement of the upper block (Figure 7.8b) usually points out a linear descending branch; if a steel tie rod is present, the coefficient α_0 that induces the activation of the kinematic mechanism increases (due to the initial pull in the tie rod) and the curve is characterized by a further increase (due to the increasing of the pull) and by the failure of the tie rod (with the return to the behaviour of the arch without the tie rod).

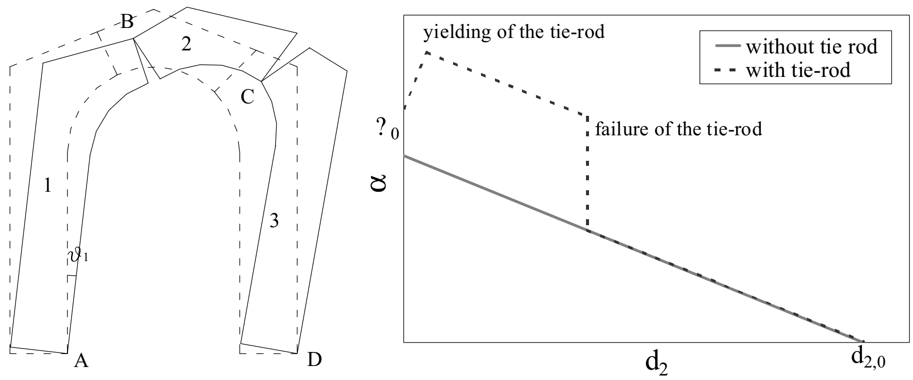

Fig. 7.8. Mechanical model for the triumphal arch in the churches: a) three blocks kinematic mechanism; b) typical pushover curves, with and without a steel tie rod

The application of the capacity spectrum method requires the transformation to an equivalent single degree of freedom system; to this aim, a well -known procedure uses a vector of nodal displacements (usually the fundamental modal shape), normalized to the value 1 in the control point (the one used for the pushover curve). In this case, the vector of the virtual horizontal displacements δ_x, used in (7.16), may be used, as it represents the mode of failure. The transformation factor is:

$$\Gamma = \delta_{x,k} \frac{\sum_{i=1}^{n+m} W_i \delta_{x,i}}{\sum_{i=1}^{n+m} W_i \delta_{x,i}^2} \tag{7.18}$$

where $\delta_{x,k}$ is the virtual displacement of the control point k (due to normalization). The spectral displacement of SDOF system is then:

$$S_d = \frac{d_k}{\Gamma} = \frac{d_k}{\delta_{x,k}} \frac{\sum_{i=1}^{n+m} W_i \delta_{x,i}^2}{\sum_{i=1}^{n+m} W_i \delta_{x,i}} \tag{7.19}$$

while the spectral acceleration is:

$$S_a = \frac{\alpha g \delta_{x,k} \sum_{i=1}^{n+m} W_i}{\Gamma \sum_{i=1}^{n+m} W_i \delta_{x,i}} = \frac{\alpha g \sum_{i=1}^{n+m} W_i \sum_{i=1}^{n+m} W_i \delta_{x,i}^2}{\left(\sum_{i=1}^{n+m} W_i \delta_{x,i} \right)^2} = \frac{\alpha g}{e_m} \tag{7.20}$$

where g is the gravity acceleration and e_m is the fraction of the mass that participates in the kinematic mechanism.

The capacity curve obtained, by transforming the seismic coefficient α and the control displacement d_k of the pushover curve (7.17) by (7.19) and (7.20), disregards the deformability of the macroelement that is involved in the collapse mechanism, as it is considered made by rigid blocks. Hence, an estimate of the vibration period T, associated to the mechanism in the phase preceding its activation, must be done. To this aim, the f.e.m. is very useful, even with linear elastic models (proper values of the modulus of elasticity should be used, in order to take into account the stiffness degradation due to micro-cracking). Non-linear finite element models may be useful also to verify and suggest the correct choice of the kinematic mechanism, as the proposed approach is reliable if the most vulnerable mechanism is selected; on the contrary, as already stated, it is very difficult to manage non-linear f.e.m. for a pushover analysis, particularly in a vulnerability analysis at territorial scale. When no specific evaluation of the period T is possible (for instance in the case of a local mechanism with a multiple connection to the rest of the building), a reference value T=0.1s may be assumed.

Thus, the capacity may be assumed as a bilinear curve:

$$\begin{aligned} S_a &= \frac{4\pi^2}{T^2} S_d && S_d \le d_y \\ S_a &= \frac{\alpha_0 g}{e_m}\left(1 - \frac{\Gamma}{d_{k,0}} S_d\right) && S_d > d_y \end{aligned} \tag{7.21}$$

where d_y is the yielding spectral displacement, given by:

$$d_y = \left(\frac{\Gamma}{d_{k,0}} + \frac{4\pi^2 e_m}{T^2 \alpha_0 g}\right)^{-1} \tag{7.22}$$

In order to implement a mechanical approach at territorial scale, it is necessary to define the collapse spectral displacement. Even if the dynamic equilibrium is theoretically possible all along the softening branch of the capacity curve, the displacement demand increases tremendously when the capacity comes down to a certain rate of the initial one. According to the results of many non-linear dynamic analyses, performed by different authors (Doherty et al., 2002; Restrepo-Vélez and Magenes, 2004) collapse is assumed at 40% of the spectral displacement $d_{k,0}/\Gamma$ that corresponds to the zeroing of the spectral acceleration capacity. In the case of bilinear capacity curve (7.21), the ductility is expressed by:

$$\mu = 0.4\left(1 + \frac{4\pi^2 e_m d_{k,0}}{T^2 \alpha_0 g \Gamma}\right) \tag{7.23}$$

Thus, the damage state thresholds $S_{d,k}$ (k=1,2,3,4) may be defined analogously to what has been done in § 7.3.2.1 for level 0 and 1 approaches (Table 7.3), except for the moderate damage (D_k=2). The evaluation of the fundamental period T is, indeed, rather uncertain and the ductility resulting from (7.23) is usually very high, as these local collapse mechanisms are related to the loss of equilibrium (overturning) rather than to the failure of masonry. Thus, it is better to define $S_{d,k}$=max(1.5d_y, 0.15μd_y).

As these local collapse mechanisms are related to the loss of equilibrium (overturning) rather than to the failure of masonry, the ductility resulting from (7.23) is usually very high. However, it is worth noting that the use of equilibrium limit analysis for the evaluation of the capacity curve may lead to unreliable results, if the validity of the hypotheses is not properly checked. The a-priori selection of the kinematic mechanism is the first critical aspect, as only plausible mechanisms should be considered; to this end, the seismic damage observation may give valuable advice. Regarding the assumption of no masonry tensile strength, it should be noted that this represents the actual behaviour in the mortar joint planes, while on a generic masonry plane a certain tensile strength should be assumed due to the interlocking among stone or brick elements; the singling out of the mechanism, by separation of the macroelement into parts, should consider planes where the tensile strength is really negligible (for instance, in the case of a façade overturning, if the interlocking between the façade and the lateral walls is good, a wedge of the latter ones should be considered in the mechanism, which contributes to stability). Contrary to this problem, which produces results on the safe side, the limited compressive strength determines lower values both of the seismic coefficient α_0 and of the zeroing control displacement $d_{k,0}$; in order to take into account this problem, the hinges that connect each couple of blocks may be placed in a rear position with respect to the border, considering a stress block behaviour; the shift of the hinge should be related to the quality of masonry and the level of initial stress on that plane.

The evaluation of the performance point (target displacement) cannot be made with the approach adopted in section 7.3.2, which considers an inelastic demand spectrum, because the cyclic behaviour of the kinematic mechanism is non-linear elastic, with no significant hysteretic dissipation. Thus, the target displacement may be obtained by the elastic demand spectrum (5% damping), considering a properly defined equivalent secant stiffness. A statistical analysis on the results of non-linear dynamic analyses with different earthquakes shows that at collapse, when the displacement demand is close to μd_y, the equivalent period may be defined on the capacity curve considering the point in which S_d=0.5μd_y; it is evident that in the linear range, the elastic period T has to be used. The equivalent period T*, useful for the evaluation of the target displacement S_{d*}, is defined by a linear interpolation between the two above mentioned situations, and the target displacement may be obtained by an iterative procedure:

$$S_{d*} = S_{de}(T^*) = \frac{T^{*2}}{4\pi^2} S_{ae}(T^*) \tag{7.24}$$

$$T^* = 2\pi \sqrt{\frac{\kappa S_{d*}}{S_a(\kappa S_{d*})}} \tag{7.25}$$

where: S_{de} is the elastic displacement demand spectrum, S_a is the capacity curve (7.21) and the coefficient κ is given by:

$$\kappa = \frac{\mu - 0.5(S_{d*} / d_y + 1)}{\mu - 1} \tag{7.26}$$

Once the target displacement S_{d*} is obtained, the lognormal fragility curves (7.9) give the probability that the damage is equal to or higher than a certain state D_k.

Figure 7.9 shows the entire procedure for the definition of the capacity curve and the evaluation of the target displacement (performance point). Figure 7.9a depicts the transformed pushover curve, the linear elastic branch, the damage state thresholds and two different demand spectra. Figure 7.9b highlights the performance point evaluation.

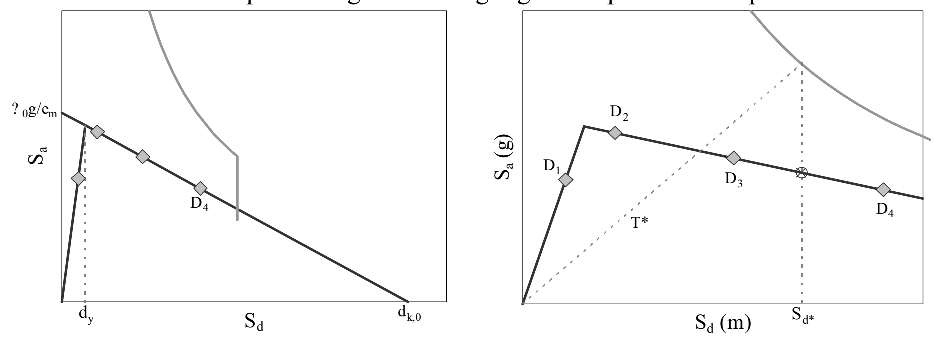

Fig. 7.9. Capacity curve and target displacement evaluation

As regards the demand spectrum, it is worth noting that usually local mechanisms in the macroelements interact with the rest of the building and in many cases they are located at a certain level, quite far from the soil foundation (for instance, the gable overturning in the church façade or the collapse of the belfry in the tower). Only in the case of macroelements which are founded on the soil and behave independently, the site response spectrum can be used. In the other cases, the demand spectrum is defined starting from the seismic coefficient proposed in Eurocode 8 (EC8, 2003) for non-structural elements, as it considers that, at the base of the macroelement, the dynamic excitation is amplified according to the dynamic properties of the whole building and the position with respect to the base:

$$S_{ae}(T^*) = a_g \left[\frac{3(1 + z/H)}{1 + (1 - T^*/T_1)^2} - 0.5 \right] \tag{7.27}$$

where: z is the height of the macroelement above the soil foundation (usually it is referred to the base of the macroelement or to its barycentre, when it is connected to the building in different points); H is the building height; T_1 is the fundamental vibration period of the building in the direction of the kinematic mechanism.

This spectrum is not valid for the high periods, because it has been proposed for the elastic verification of non-structural elements, which usually are quite stiff; on the contrary, the use with the capacity spectrum method requires reliable values also for the high periods. Thus, for $T^*>1.5T_1$, the spectrum is extended with the constant velocity trend ($S_{ae}(T^*)\propto 1/T^*$). Moreover, in the case of flexible buildings with respect to the characteristic period of the soil ($T_1>T_C$), this spectrum overestimates the demand, as it doesn't consider the reduction of the seismic amplification in the building; to this end, a correction factor $\zeta(T^*)$ is applied to (7.27), which reduces the spectrum consistently (for $T^*=T_1$ the reduction is T_C/T_1):

$$\zeta(T^*) = \begin{cases} 1 & T_1 \le T_C \\ \left[1 + \left(\frac{T_1}{T_C} - 1\right)\frac{T^*}{T_1}\right]^{-1} & T_1 > T_C \end{cases} \tag{7.28}$$

In Figure 7.10a the demand spectrum for two macroelements at different heights is compared with a typical site spectrum (T_1=0.3s, T_C=0.5s): if the macroelement is close to the foundation (z/H=0.2), the demand spectrum is similar to the site dependent spectrum, filtered by the building fundamental period, while, if the macroelement is on top of the building, the amplification is huge. In Figure 7.10b the demand spectra for a macroelement at medium height (z/H=0.4) are compared, in the case of different buildings (T_1=0.15/0.3/0.6 s). If the building fundamental period is not known, a value $T_1=0.8T_C$ may be assumed.

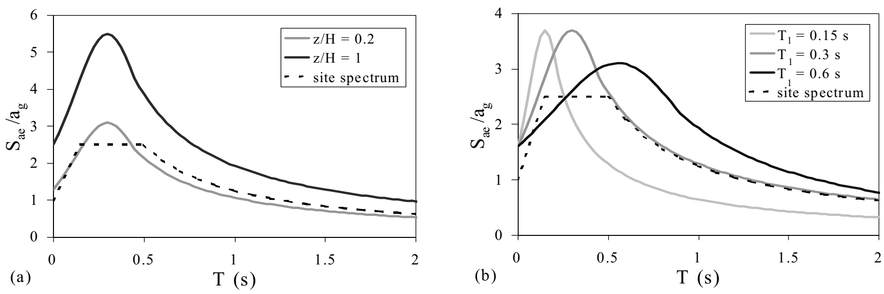

Fig. 7.10. Acceleration demand spectra for macroelements: a) influence of the height (T_1=0.3s) ; b) influence of the fundamental building period (z/H=0.4)

The target displacement S_d^* may be obtained by the iterative procedure (7.24-25-26), by the following displacement demand spectrum:

$$\begin{aligned} S_{de}(T^*) &= \frac{a_g\zeta(T^*)T^{*2}}{4\pi^2}\left[\frac{3(1+z/H)}{1+(1-T^*/T_1)^2} - 0.5\right] && T^* < 1.5T_1 \\ S_{de}(T^*) &= \frac{1.5a_g\zeta(T^*)T_1T^*}{4\pi^2}\left(1.9 + 2.4\frac{z}{H}\right) && 1.5T_1 < T^* < T_D \\ S_{de}(T^*) &= \frac{1.5a_g\zeta(T_D)T_1T_D}{4\pi^2}\left(1.9 + 2.4\frac{z}{H}\right) && T^* > T_D \end{aligned} \tag{7.29}$$

7.5.1. EXAMPLES AND APPLICATIONS

The level 2 mechanical approach has been widely applied in the RISK-UE Project, *An advanced approach to earthquake risk scenarios* (Mouroux et al., 2004).

The vulnerability of the churches in Catania (ITA) has been assessed with reference to the overturning of the façade and the collapse mechanism of the triumphal arch (Lagomarsino et al., 2004b).

A very interesting example of application of the limit analysis to monumental buildings is the vulnerability assessment of Santa Maria del Mar church in Barcelona, Spain (Irizarry et al., 2004). The seismic transversal response of the nave was analysed by extracting a macroelement corresponding to a central bay (Figure 7.11) and performing both finite element analyses (modal and pushover) and the seismic limit analysis.

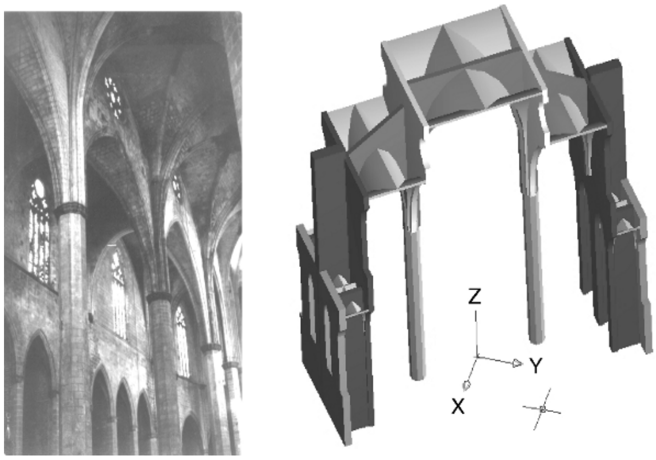

Fig. 7.11. An internal view of the nave of Santa Maria del Mar church and the macroelement extracted for the analysis of the seismic transversal response

The fundamental period obtained by the modal analysis is T_1=0.8s; the pushover analysis allows to single out the collapse mechanism, by the position of the cracked areas (Figure 7.12), obtained by a proper non -linear constitutive model for masonry (Calderini and Lagomarsino, 2004). The kinematic mechanism is made by ten rigid blocks and seven hinges; the weight and the centroid of each block are obtained by a CAD 3D geometrical model; Figure 7.12 shows the collapse mechanism, for a finite value of the incremental displacement.

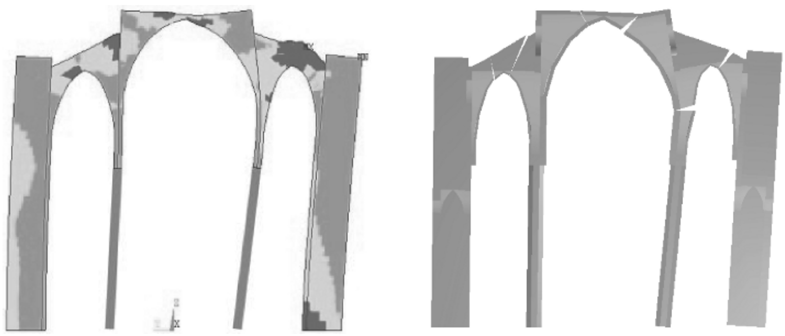

Fig. 7.12. Cracked zones (in red) obtained by a finite element pushover analysis and collapse mechanism analysed by the equilibrium limit approach

The capacity curve is shown in Figure 7.13, together with the damage state thresholds. A probabilistic hazard analysis of Barcelona has provided, for the proper site conditions, the demand spectrum (a_g=0.15 g, T_C=0.4 s); the target displacement, obtained with the procedure (7.24) and (7.25), is lower than $S_{d,2}$, thus the probability of suffering a moderate damage is less than 50%.

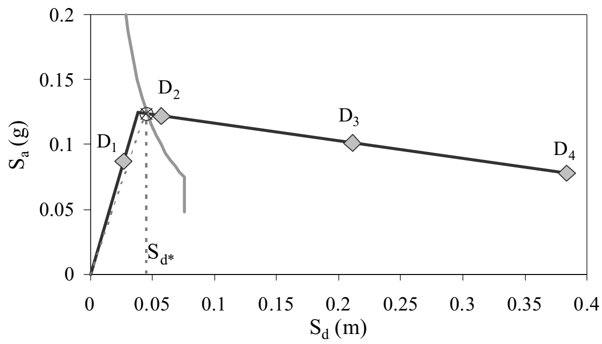

Fig. 7.13. Capacity curve of the transversal response of the nave, with the damage state thresholds, and evaluation of the target displacement

7.6. Final remarks

The vulnerability of the cultural heritage may be assessed also at territorial scale, with the aim of planning preventive mitigation strategies. Both observational and mechanical based methods have been stated, which have to be used respectively with intensity and PGA hazard scenarios. The methods are more or less detailed, depending on the level thoroughness of the available information.

The level 0 methodologies may be applied using poor data, even with a simple list of monuments, and provides a general estimation of the vulnerability of the monumental heritage, on the base of the typology, in a wide seismic area or in a big city.

The level 1 methodologies require a quick survey for the appraisal of vulnerability parameters in each structure; it results in a differentiation among monuments of the same typology, which is useful for preventive mitigation strategies and emergency planning.

The level 2 methodologies consist of a deep analysis of the single monument, by the survey of the most vulnerable macroelements. Besides the typical use in a seismic risk analysis, as for level 1 methods, these results may influence the effectiveness of different retrofitting interventions, applied systematically for the risk mitigation.