Chapter 1

Seismic Vulnerability of Existing Buildings: Observational and Mechanical Approaches for Application in Urban Areas

1.1. Introduction

Past and recent earthquakes have shown the high level of seismic vulnerability of old and historic down-town areas: the 2009 L'Aquila earthquake is one of the latest dramatic examples, in which several historical centers (such as – besides L'Aquila – Onna, Castelnuovo and Villa Sant'Angelo) were severely affected, with heavy damage extended across whole built-up areas and the collapse of large portions (sometimes even in their totality) of many urban blocks. This follows the relevance of providing reliable vulnerability and risk analyses from the economic, cultural and human safety points of view.

As known, vulnerability represents the intrinsic predisposition of the building to be affected and suffer

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damage as a result of the occurrence of an event of a given severity. The main aims of a vulnerability analysis on a large scale – such as that of a town – are (1) to be aware of the impact of an earthquake to groups of buildings in the area; (2) to plan preventive interventions for the seismic risk mitigation; and (3) to help the management of the emergency after a major earthquake.

The main steps of a vulnerability analysis may be summarized as follows:

1) acquisition and examination of the data available in the area of interest, identification of building classes and definition of the related *vulnerability models*;

2) for each class, the definition of building parameters which models are based on; according to the data available, the parameters set can be single or differentiated for a micro-area;

3) partition of the territory into a number of zones, each characterized by a uniform *hazard*; disaggregation of the exposure data into different classes homogeneous for vulnerability;

4) for each building class and micro-area, evaluation of the performance point, fragility curves and damage probabilities (taking into account – less or more accurately – the uncertainties involved).

Vulnerability models are the tools to establish a correlation between a hazard and structural damage. As a function of the model adopted, a *hazard* may be represented in terms of the macroseismic intensity, peak ground acceleration (PGA) or the response spectrum. Structural damage is usually classified into various levels depending on the seriousness and extent in buildings; thus, building performance levels (PL) (i.e. immediate occupancy, damage control, safety to life and collapse prevention) may be associated with selected damage levels, on the basis of the consequences related to the advisability of post-earthquake occupancy, the risk to the safety of life or the ability of the building to resume its normal function. The structural damage is the cause of many other losses expected after an earthquake. Economic losses and consequences to buildings (unfit for use and collapsed buildings) and inhabitants (homelessness and casualties) can be estimated after physical damage has been determined. To this end, many statistical correlation laws, translating structural damage into percentage of losses, are proposed in the literature.

On a large scale, since usually the available data are not sufficient to define detailed models, vulnerability models cannot be applied building-by-building: thus, the vulnerability assessment has to refer to a building stock characterized by homogeneous behavior. In this sense, the evaluation assumes a statistical meaning that is consistent to the purposes of a risk analysis, that is to evaluate the probability having certain consequences on the examined area.

Several methods for the vulnerability assessment have been developed and proposed in recent years, which are implemented with the different kind of data (from poor statistical data about the building type and the number of floor to data specifically surveyed for seismic vulnerability assessment). They are based on various approaches, which may be basically classified according to the following two classes: the *macroseismic* (or observational) and the *mechanical* models.

Macroseismic models are derived and, consequently, calibrated from damage assessment data, collected after earthquakes in areas that suffered different intensities. Considering a set of buildings with a homogeneous behavior, damage is described by damage probability matrices (DPM); DPM traditionally are associated with a discrete number of building classes. Thus, the lack of information relative to damage grades for all levels of intensity, at a given geographical location or region characterized by a given building stock type, may lead to incomplete matrices; usually to complete the matrices for non-populated levels of damage and intensity binomial coefficients are used. In order to pass from discrete to continuous vulnerability evaluation, as proposed as an example in Giovinazzi and Lagomarsino [GIO 04], proper fragility curves may be introduced to correlate the intensity to the mean damage grade μ_D (a continuous parameter, $0 < \mu_D < 5$), and a histogram of damage grades is evaluated by a proper discrete probabilistic distribution (binomial). The fragility curve is defined by two parameters, the vulnerability index and a ductility index, which should be evaluated from the information about the building.

Mechanical models describe the structural response by means of a force–displacement curve, called *capacity curve*, representative of the equivalent inelastic single degree of freedom (SDOF) system; this curve provides essential information in terms of stiffness, overall strength and ultimate displacement capacity. In the case of vulnerability assessment at large scale, this curve aims to idealize the response of an entire stock of structures with homogeneous behavior. Assuming a bilinear form without hardening, three quantities basically need to be defined; different choices, as clarified in the following sections, may be adopted in selecting the independent and derived entities. This curve idealizes the response which could be achievable by subjecting the structure, idealized through an adequate model, to a static horizontal load pattern of increasing amplitude, aimed at describing the equivalent seismic forces: thus, it establishes a relationship between the demand and the structural capacity. Each point of this curve is associated with an exact pattern and level of damage (Figure 1.1). The expected damage assessment is provided by comparing the "capacity curve" with the "seismic demand", in the form of a response spectrum (resulting from codes recommendations or more sophisticated hazard analyses). This approach is coherent with the current trend of nonlinear static procedures for the evaluation of the seismic performance of masonry buildings (e.g. the capacity spectrum method and the N2 Method). Finally, by defining proper damage levels (corresponding to predefined displacement values) on the capacity curve it is possible to evaluate the distribution of damage levels (and thus a mean damage index). The application to the large scale requires that these models are based on a limited number of geometrical and mechanical parameters. This need implies that mechanical models have to be in some way "simplified"; moreover, their application to the assessment of existing buildings, often designed following empirical rules of art (especially in the case of masonry constructions), may be in some cases conventional when the principles and rules of the design approach inspire the formulation of these models. An alternative for the definition of these curves could be by referring to detailed numerical analyses provided on prototype buildings, for which an accurate geometrical and mechanical characterization is available; however, the extrapolation of the results obtained on a single building to the entire corresponding stock may be quite conventional, with the drawback of not being able to exactly quantify the response variability associated with the uncertainties of parameters. Unlike the case of macroseismic models, which are calibrated on the basis of an earthquake damage survey, the validation of the mechanical models represents an issue much more complex since this direct comparison is not available. A possible alternative is to compare the results of the mechanical models to those provided by the macroseismic models; this comparison requires the introduction of suitable correlation laws between the parameters of the hazard (i.e. PGA and intensity).

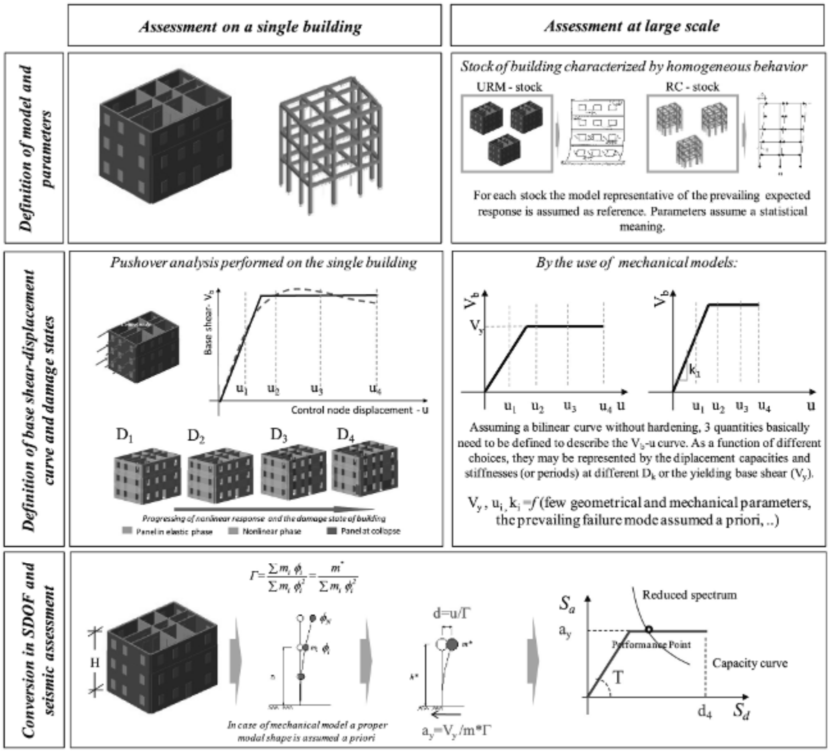


Figure 1.1. *Force–displacement curve obtained in case of pushover analysis performed on a single structure or by a mechanical model applied at large scale*

A cross-validation of two approaches is proposed in Lagomarsino and Giovinazzi [LAG 06b].

Once the model adopted as the reference is defined, the first step of the above-mentioned methodology consists of processing the available data in order to aggregate them for homogeneous behavior classes and defining proper values for the model parameters (representative of each building class). Usually the following factors are considered: structural material (masonry and reinforced concrete, RC); structural system (i.e. pilotis; RC frame building, with or without infilled panels efficiently connected; masonry buildings with RC beams coupled to spandrel elements or with weak spandrels); number of stories; and age. In particular, the age is important for the choice of codes to assume as the reference in order to define the basic design principles according to which the building stock has been designed. To define the necessary parameters, it is possible to operate in different ways as a function of the data already available. Obviously, if databases (which already contain all the necessary data) have been arranged, it is possible to directly proceed to the statistical evaluation of the parameters. However, since in most of the cases the entire set of the necessary data is not available, usually reference is made to both the sample survey provided on the building representative of the selected classes and to data available in the literature on similar stocks.

In the following sections, after some explanation on the damage levels and the buildings type of classification, some models to be used for vulnerability analyses are presented considering both the above-mentioned approaches (the macroseismic and mechanical approaches) and both unreinforced masonry (URM) and RC buildings. In particular, in the case of the macroseismic approach, the model proposed in Lagomarsino and Giovinazzi [LAG 06b] is discussed, and in the case of the mechanical approach the displacement-based vulnerability (DBV)-*concrete* and DBV-*masonry* methods proposed in Lagomarsino *et al.* [LAG 10] have been adopted as the reference; the DBV-*masonry* method starts from the model originally proposed in Cattari *et al.* [CAT 05], whereas the DBV-*concrete* starts from the displacement-based earthquake loss assessment (DBELA) method proposed in Crowley *et al.* [CRO 04]. In both cases, starting from the original formulations, some significant modifications have been introduced by the authors as discussed in following sections.

1.2. Damage levels and building types classification

Vulnerability models establish a correlation between hazard and structural damage for a building stock characterized by homogeneous behavior. Thus, firstly it needs to define the damage levels (D_k, with k = 1, …, 5) and the building types classification to be adopted as a reference. To this end, the classification proposed in the European Macroseismic Scale EMS-98 [GRU 98] is adopted; it has been implemented with some slight modifications proposed in Lagomarsino and Giovinazzi [LAG 06b], related to the classification of structural systems.

EMS-98 proposes five discrete levels corresponding to the occurrence of slight (D_1), moderate (D_2), heavy (D_3), very heavy (D_4) and destruction (D_5) damage grades, respectively. In particular, damage levels may be differentiated as a function of the seriousness and the extent of the damage that occurred in structural elements. Regarding this, structural damage is adopted as the main parameter to be considered, even if non-structural damage can be equally important for the loss evaluation. It can be argued [KIR 97] that non-structural damage can be more drift or acceleration sensitive; however, since the detailed assessment of non-structural damage may result, that is, quite problematic and conventional by models (usually it is related to the attainment of inter-story drift values), it seems preferable than mainly referring to the structural damage. Moreover, a specific performance building level may be associated with each damage level. In particular, a more detailed description of the above-mentioned damage levels may be summarized as follows:

1) D_1: no damage, either structural or non-structural; the expected response is essentially linear elastic, yielding is not attained in any critical section. With reference to the performance level expected, the building is immediately usable after the earthquake;

2) D_2: minor structural damage and/or moderate non-structural damage; yielding condition is attained in many critical sections. With reference to the performance level expected, in most of the cases, the building can be utilized after the earthquake without any need for significant strengthening and repair to structural elements.

3) D_3: significant structural damage and extensive non-structural damage. With reference to the performance level expected, in most of the cases the building cannot be used after the earthquake without significant repair. Still repairing and strengthening are feasible.

4) D_4: state next to the collapse. With reference to the performance level expected, usually, repairing the building is neither possible nor economically reasonable.

5) D_5: state of ruin.

Macroseismic models, since they are calibrated on observed data, usually directly refer to this five-level graduated damage scale. On the contrary, mechanical models imply conventional and suitable rules to define damage levels on the capacity curve. In particular, they may be related to selected values of displacement capacity: in fact, according to the achievement of performance-based earthquake engineering, it is generally assumed that a better description of the progressing of the nonlinear response of the structure may be provided in terms of displacement rather than strength. However, in the case of a mechanical model, making a clear distinction between very heavy damage (D_4) and a state of ruin (D_5) is very hard: in fact, it is clear that, while by a visual inspection (as in the case of a damage scale based on the post-earthquake damage survey) it is easy to distinguish what is collapsed from what is heavily damaged (but still standing), the same distinction is not possible on a curve, when the structure has lost its static equilibrium condition (as summarily indicated in Figure 1.2). As a consequence, in the case of mechanical models, only the first four damage levels are defined on the capacity curve; thus, with reference to the losses assessment, further relationships may be introduced in order to overcome this partial inconsistency between the damage state definition usually adopted in mechanical and macroseismic models (as discussed in Lagomarsino and Giovinazzi [LAG 06b]). As a function of the mechanical models adopted, the displacement capacity values which mark the transition from one damage level to the other may be defined, respectively: (1) directly on a mechanical basis and (2) as a function of the global ductility as a proper ratio of it. As an example, in the case (1), displacement capacities may be associated with chord rotation or drift values of the main structural element which governs the global response. Once damage levels have been defined, for aims of seismic assessment, they could be correlated by proper “acceptability thresholds” to corresponding performance levels. Usually, the basic assumption is to assume as “acceptability criteria” that PLs are coincident with the corresponding damage levels: this assumption is also adopted in the following. Indeed, damage levels represent discrete damage conditions: from a probabilistic point of view it is usually assumed that these thresholds correspond to the conditional probability of 50% of being in or exceeding the corresponding damage level. It means that in correspondence with the displacement d_k there is a “small” probability of having damage levels higher than k-th, with consequences that could be incompatible with the fulfillment of the examined PL: as a consequence, in general, to assume PLs as strictly coincident with the corresponding damage levels could not always be on the safe side. A refinement of the definition of PLs could be obtained by introducing a probabilistic approach through fragility curves and by checking the fulfillment of some acceptance criteria (e.g. defined in terms of an *admissible* percentage of collapsed buildings or injured people).

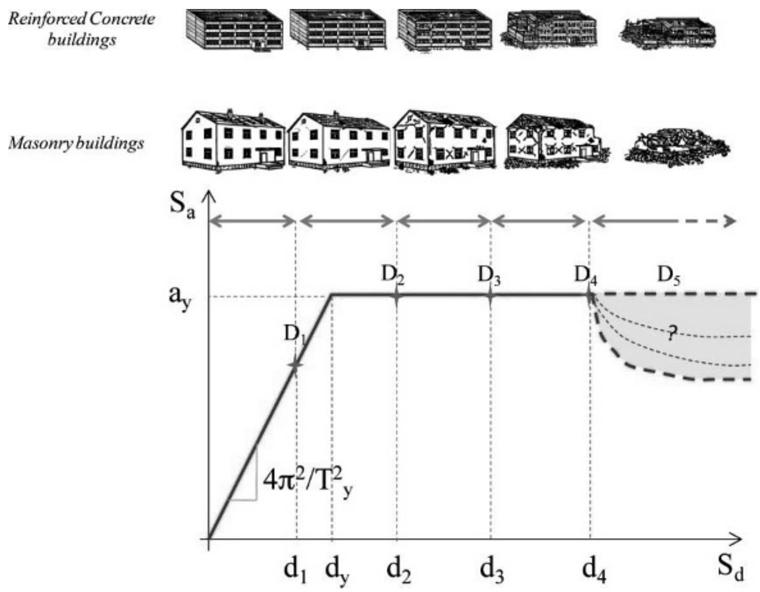


Figure 1.2. *Possible definition of the damage levels on the capacity curve*

As regards the buildings typological classification, reference is made to that proposed in Lagomarsino and Giovinazzi [LAG 06b] and already also adopted in the Risk-UE project [MOR 05]. Table 1.1 summarizes this classification.

Unreinforced masonry		Reinforced/confined masonry	
M1	Rubble stone	M7	Reinforced/confined masonry
M2	Adobe (earth bricks)		
M3	Simple stone		Reinforced concrete
M4	Massive stone	RC1	Concrete moment frame
M5	U masonry (old bricks)	RC2	Concrete shear walls
M6	U masonry – RC floors	RC3	Dual system

Table 1.1. *Building types classification*

This classification system essentially corresponds to the system adopted by EMS-98, apart from the inclusion of reinforced concrete dual system typology RC3. Moreover, it is possible to introduce sub-typologies. In particular, the type of horizontal structure has been considered for masonry buildings: wood slabs (from $M1_w$ to $M7_w$), masonry vaults (from $M1_v$ to $M7_v$), composite steel and masonry slabs (from $M1_{sm}$ to $M7_{sm}$) and reinforced concrete slabs (from $M1_{ca}$ to $M7_{ca}$). Pilotis sub-typology (from $RC1_p$ to $RC3_p$) has been introduced to take into consideration, for all the RC typologies, vertical irregularity, often leading to soft-story collapse mechanisms, while the presence of effective infill-walls has only been considered for reinforced concrete frame typology ($\mathrm{RC1}_i$). In the case of the mechanical model described in section 1.4, only classes from M1 to M6 and RC1 are examined. For all the building types, different classes of height are considered. In the case of the macroseismic model, three classes of height have been considered (*L*=…) differently defined in terms of floor numbers for masonry (*L*=…) and reinforced concrete buildings (*L*=…). In the case of the mechanical model, the inter-story and total height parameters are explicitly taken into account. Moreover, for buildings designed according to a seismic code, it has been considered: the level of the seismic action depending on the seismicity (*I* = zone I, *II* = zone II, *III* = zone III); the ductility class, depending on the prescription for ductility and hysteretic capacity (*WDC* = without ductility class, *LDC* = low ductility class, *MDC* = medium ductility class and *HDC* = high ductility class).

1.3. The macroseismic approach

The macroseismic model described in the following refers to that originally proposed in Giovinazzi and Lagomarsino [GIO 01] and further improved in Lagomarsino and Giovinazzi [LAG 06b].

The vulnerability is measured in terms of a vulnerability index *V* and a ductility index *Q*, both evaluated taking into account the building type and its constructive features. A hazard is described in terms of macroseismic intensity, according to the European Macroseismic Scale EMS-98, which is considered, in the framework of the macroseismic approach, as a continuous parameter evaluated with respect to a rigid soil condition; possible amplification effects due to different soil conditions are accounted for through a modification of the vulnerability parameter *V*.

The correlation between the seismic input and the expected damage, as a function of the assessed vulnerability, is expressed in terms of fragility curves (Figure 1.3(a)) described by a closed analytical function:

$$\mu_D = 2.5\left[1 + \tanh\left(\frac{I + 6.25V - 13.1}{Q}\right)\right] \qquad [1.1]$$

where *I* is the macroseimic intensity (seismic input) and *V* and *Q* are, respectively, the vulnerability and the ductility indexes.

Equation [1.1] allows the estimation of the mean damage value μ_D ($0 < \mu_D < 5$) of the expected discrete damage distribution (Figure 1.3(b)):

$$\mu_D = \sum_{k=0}^{5} p_k k \qquad [1.2]$$

The probabilistic assessment, in terms of both damage distributions and fragility curves (Figure 1.3(b)), for the mean damage value μ_D evaluated according to equation [1.1], is obtained assuming a binomial distribution. Therefore, the probability p_k of having each damage grade D_k (k = 0, …, 5), for a certain mean damage μ_D, is evaluated according to the probability mass function of the binomial distribution:

$$p_k = \frac{5!}{k!\,(5-k)!}\left(\frac{\mu_D}{5}\right)^k\left(1-\frac{\mu_D}{5}\right)^{5-k} \qquad [1.3]$$

where ! indicates the factorial operator.

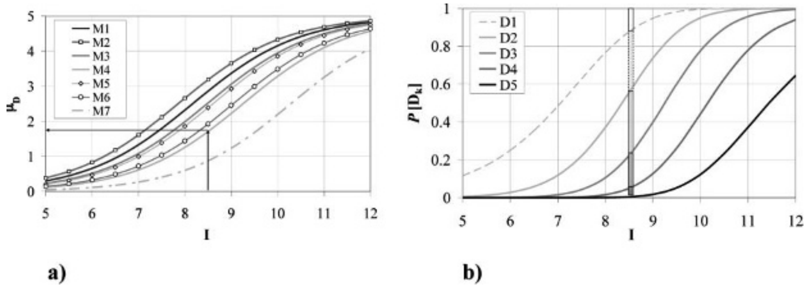


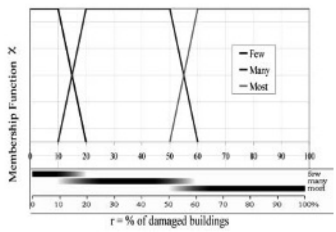
Figure 1.3. *Macroseismic method: a) fragility curves for different masonry building types; expected damage μ_D = 1.7 for M4 typology when I = 8.5; (b) fragility curves for the building typology M4 as a function of I; damage distribution for I = 8.5 (from Giovinazzi and Lagomarsino [LAG 06])*

The binomial distribution has been adopted for the macroseimic approach, being successfully used for the statistical analysis of data collected after the 1980 Irpinia (Italy) earthquake [BRA 82]. A non-negligible critical aspect with the binomial distribution is that it does not allow defining a different scatter around the mean value μ_D. For this reason, a beta distribution may be used. In particular, in Giovinazzi and Lagomarsino [GIO 05b], defining parameters of the beta function has been suggested, on the basis of a fuzzy-random approach, in order to have different scattered damage distributions, depending on the amount of the cognitive uncertainties affecting the vulnerability assessment.

This macroseimic method has been originally derived by the Giovinazzi and Lagomarsino [GIO 01] from the EMS-98 macroseimic scale and verified and calibrated on the basis of damage data from different earthquakes. In particular, the EMS-98 scale groups together buildings into six vulnerability classes, from A to F, at decreasing vulnerability. The frequency of the expected damaged is defined by linguistic terms ("few", "many" and "most") considering five damage grades D_k (k = 1, …, 5).

I	D_1	D_2	D_3	D_4	D_5
V	Few				
VI	Many	Few			
VII			Many	Few	
VIII				Many	Few
IX					Many
X					Most
XI					
XII					

a)



b)

Figure 1.4. *EMS-98 macroseimic scale: a) implicit damage probability matrix (DPM) for class A; b) linguistic terms, "few", "many" and "most" described by the EMS-98 scale as overlapping frequency intervals and interpreted in terms of membership functions (from Giovinazzi and Lagomarsino [LAG 06])*

From Figure 1.4(a) it can be noted how the definitions provided by the EMS-98 scale might be regarded as implicit DPM. In order to numerically translate and complete these implicit DPM, Giovinazzi and Lagomarsino [GIO 01] proposed the combined use of the fuzzy set theory and the probability theory. Values suggested by the scale in a graphical fuzzy manned as overlapping intervals of frequencies in the range 0–100 (Figure 1.4(b)) have been assumed for the linguistic qualitative definitions "few", "many" and "most".

According to the fuzzy set theory [DUB 80], the overlapping intervals of frequencies have been mathematically described as trapezoidal membership functions χ (Figure 1.4(b)), attributing to a complete and a full membership to the definite ranges $\chi(r) = 1$ (few $0 \leq r \leq 10$, many $20 \leq r \leq 50$, most $60 \leq r \leq 100$) and representing by overlapping ranges $0 < \chi(r) < 1$ the elements that do not have a complete membership to one of the provided definitions (i.e. the boundary between "few" and "many" $10 \leq r \leq 20$). The membership functions have been translated into a crisp set applying an α-cut procedure $R_\alpha=\{r|\chi(r) \geq \alpha\}$. In particular, reference has been made to the cuts $\alpha = 0$ and $\alpha = 1$ (i.e. for $R_{\alpha=0} = \{0, 20\}$, $R_{\alpha=1} = \{0, 10\}$ for the "few" membership function). The bounds of the crisp set identified by the cut $\alpha = 1$ (labeled as + and – in Figure 1.5(a)) have been assumed as reference values to derive the probable behavior for each one of the class, while the bounds of the crisp set $\alpha = 0$ (labeled as + + and – – in Figure 1.3(a)) have been assumed to bound behaviors that are less probable, but that could be still possible. For both the cases the upper bound of the crisp set is representative of the more vulnerable situation, while the lower bound represents the less vulnerable behavior.

α–cuts		few	many	most
α=0 -upper	++	20	60	100
α=1 -upper	+	10	50	100
α=1 -lower	-	0	20	60
α=0 -lower	--	0	10	50

a)

I=VIII	D_0 -	D_1 -	D_2 -	D_3 -	D_4 Many	D_5 Few	μ_D
A++	0	2	11	29	38	**20**	3.6
A+	1	6	20	34	29	**10**	3.2
A-	2	12	28	33	**20**	5	2.7
A--	6	22	34	26	**10**	2	2.2

b)

Figure 1.5. *Derivation of the macroseismic method from EMS-98 implicit DPM: a) plausible and possible upper and lower values assumed for EMS-98 linguistic terms; b) damage probability distributions and mean damage values resulting from the numerical translation of the linguistic definition for class A and intensity $I_{EMS\text{-}98}$ = VIII (from Giovinazzi and Lagomarsino [LAG 06])*

Once the numerical values in Figure 1.5(a) have been assumed for the translation of the linguistic terms, reference has been made to the binomial probability density function in order to complete EMS-98 DPM. For each one of the considered conditions (++, +, –, – –), the mean damage value μ_D has been evaluated allowing the binomial distribution to provide the better approximation for the assumed numerical values. When the linguistic definitions were provided by the scale for two different damage levels (i.e. class A and I = VIII in Figure 1.5(b)), probable (+) and less probable (++) distributions representative of more vulnerable situations have been obtained assuming, as reference values, the distributions for the linguistic term "few" (Figure 1.5(a)). On the other hand, the probable (–) and the less probable (– –) distributions, representing less vulnerable situations, are obtained assuming, as reference values, the distributions associated with the linguistic term "many" (Figure 1.5(b)). The representation of the resulting mean damage values μ_D, as a function of the intensity I, has led to the definitions of fragility curves identifying distinct areas of probable behavior (bounden by + and – curves) for each vulnerability class (Figure 1.6(a)) and overlapping areas of less probable behavior (bounden by ++ and – – curves) for adjacent vulnerability classes (Figure 1.6(b)).

A conventional vulnerability index V has been introduced to represent the position of the probable and the less probable behavior areas identified in the I–μ_D diagram. As a function of this index V, an analytical function has been proposed for the fragility curve interpolation in equation [1.1] [GIO 04].

In compliance with the probable and the less probable area of behavior, identified by the fragility curves, probable and less probable vulnerability index ranges, referred to as V^{-}÷V^{+} and V^{--}÷V^{++}, respectively, have been associated with each vulnerability class. This has lead to the definition of vulnerability index membership functions, where a full membership, $\chi(V)$, has been assumed for the probable ranges of each class, while a membership, $0 < \chi(V) < 1$, has been assumed for the probable ranges of each class, and a membership, $\chi(V) = 1$ has been assumed for the overlapping ranges of values (Figure 1.7(a)). A single representative value of the vulnerability index V for each vulnerability class has been identified via the centroid defuzzification method [ROS 95]. Figure 1.7(b) shows fragility curves drawn according to the proposed analytical equation [1.1] as a function of the representative vulnerability index V values, where $Q = 2.3$.

It is worth noticing that the vulnerability index V has been conventionally defined ranging from –0.02 to 1.02. Anyhow better or worse behaviors, with respect to the less vulnerable class, F, and to the more vulnerable class, A, are accounted for within the fuzzy partition proposed for the vulnerability index (respectively represented as class X and class Y in Figure 1.7(a)).

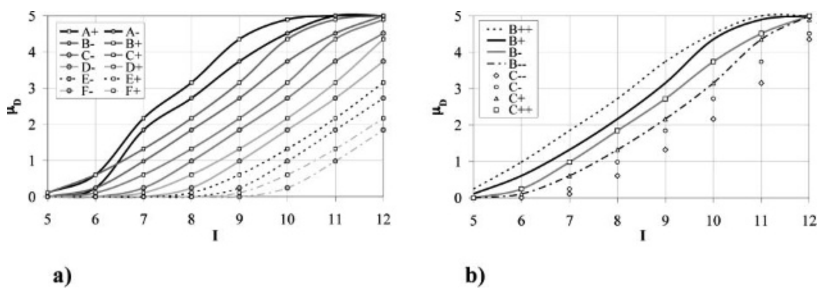
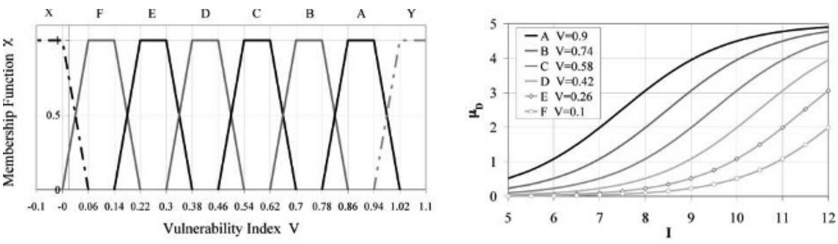


Figure 1.6. *Derivation of the fragility curves from EMS-98 implicit DPM: a) distinct areas of probable behavior for all the vulnerability classes; b) probable behaviors and overlapping areas of less probable behaviors for class B and class C (from Giovinazzi and Lagomarsino [LAG 06])*



a) **b)**

Figure 1.7. *Macroseimic method for vulnerability classes: a) membership functions $\chi(V)$ for the vulnerability index V; b) fragility curves for the vulnerability classes as a function of the representative value assumed for the vulnerability index V (from Giovinazzi and Lagomarsino [LAG 06])*

With reference to the EMS-98 vulnerability table [GRU 98], where the seismic behavior of building typologies is correlated with the seismic behavior of vulnerability classes, the definition of the macroseismic method has been extended to the buildings' typologies. As a matter of fact, the EMS-98 vulnerability table identifies for each typology, the most likely vulnerability class plus probable and less probable ranges of behaviors (Figure 1.8(a)). These linguistic judgments have been numerically translated according to the fuzzy set theory. The membership function of each building type $\chi(V)$ has been obtained by the soft union of the membership function. [ROS 95] ascribed to the vulnerability classes, each type considered with its own degree of belonging. As an example, the membership function for the building typology M4 is shown in Figure 1.8(b). Probable V^- to V^+ and less probable vulnerability index ranges V^{--} to V^{++} have been identified by α-cut procedures, respectively, for cuts $\alpha = 1$ and for $\alpha = 0.5$. For each one of the typology, a representative value V of the vulnerability index has been identified via a centroid deffuzification procedure [ROS 95].

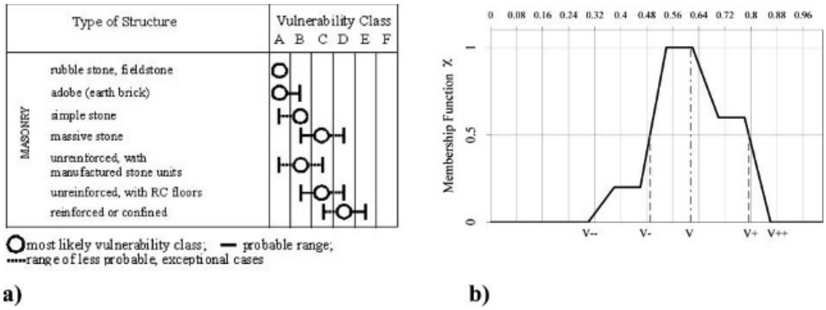


Figure 1.8. *Building typologies: a) EMS-98 vulnerability table for masonry building typologies; b) membership function $\chi(V)$ for M4 building typology (from Giovinazzi and Lagomarsino [LAG 06])*

Typologies		Building type	V^{--}	V^{-}	V	V^{+}	V^{++}
Masonry	M1	Rubble masonry	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	U Masonry (old bricks)	0.46	0.65	0.74	0.83	1.02
	M6	U Masonry – RC floors	0.3	0.49	0.616	0.79	0.86
	M7	Reinforced/confined masonry	0.14	0.33	0.451	0.633	0.7
Reinforced concrete	RC1	Frame in RC (without aseismic design.)	0.3	0.49	0.644	0.8	1.02
		Frame in RC (moderate E.R.D.)	0.14	0.33	0.484	0.64	0.86
		Frame in RC (high E.R.D.)	–0.02	0.17	0.324	0.48	0.7
	RC2	Shear walls (without E.R.D.)	0.3	0.367	0.544	0.67	0.86
		Shear walls (moderate E.R.D.)	0.14	0.21	0.384	0.51	0.7
		Shear walls (high E.R.D.)	–0.02	0.047	0.224	0.35	0.54

Table 1.2. *Vulnerability index values for building typologies*

Fragility curves for the building typologies can be drawn as a function of the vulnerability index values *V* provided in Table 1.2 and of the ductility index, $Q = 2.3$.

In order to achieve a validation of the proposed method, the fragility curves, derived for the building typologies, have been compared with observed damage data (Figure 1.9(a)) and with other observed vulnerability approaches (Figure 1.9(b)). A good agreement has been, generally, observed [GIO 05a].

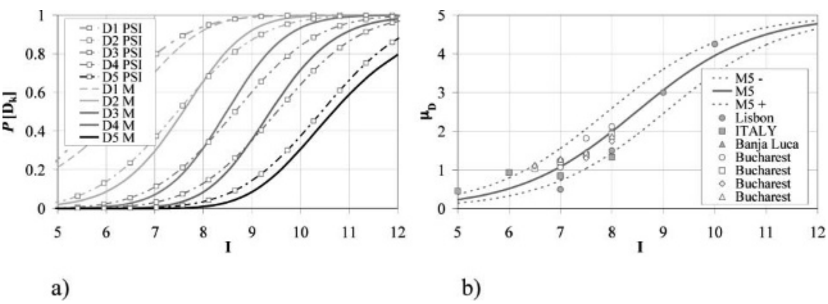


Figure 1.9. *Validation of the method for M5 unreinforced masonry (old brick): a) comparison between fragility curves from the macroseimic method and the ones from Coburn and Spence [COB 92] PSI method; b) comparison between fragility curves and observed damage data from different earthquakes (from Giovinazzi and Lagomarsino [LAG 06])*

1.4. The mechanical approach

Mechanical models describe the structural response of a system by means of a capacity curve that provides essential information in terms of stiffness, overall strength and ultimate displacement capacity.

Assuming an elastic perfectly plastic form, three quantities basically need to be defined (e.g. the ultimate strength, the ultimate displacement capacity and the elastic vibration period). Different choices, as clarified in the following, may be made in selecting the independent and derived entities; once defined, they are computed on the basis of simplified formulations based on few mechanical and geometrical parameters. If available, the comparison with experimental results and data of a real existing building (representative of the stock examined) may be particularly relevant for calibrating these entities: for example, in Michel *et al.* [MIC 10] a comparison between the vibration period computed through some simplified formula proposed in literature and codes that was obtained by ambient vibration recordings on a set of existing buildings in France is illustrated.

The mechanical approach presents the following main advantages: employing the results of sophisticated hazard analyses (by using the seismic input in the spectral form) and explicitly taking into account the different parameters which define the structural response.

The capacity curve represents the response of an equivalent inelastic *SDOF* system representative, in the case of vulnerability analyses on a territorial scale, of the response of an entire stock of buildings characterized by homogeneous behavior.

Thus, the expected seismic performance d_{PP}, or performance point, is evaluated by comparing the seismic demand, represented by properly reduced elastic spectra (by either an overdamped or inelastic approach), with the capacity curve of the equivalent *SDOF*. According to this approach, in the past decade, the achievement of performance-based earthquake engineering concepts has led to an increasing utilization of nonlinear static procedures in evaluation of the seismic performance of buildings. The *capacity spectrum method* (originally proposed by Freeman *et al*. [FRE 75] and adopted in the ATC-40 [ACT 96]) and the *N2 Method* (originally proposed by Fajfar [FAJ 00] and used in Eurocode 8 – Part 1 [EUR 05a] and in the Italian

Technical Code [ITA 08]) are the most frequently used. The *capacity spectrum method* refers to the use of overdamped spectra, whereas the *N2 Method* refers to the use of the inelastic spectra.

Once the performance point d_{PP} is evaluated and the proper damage states are defined on the capacity curve d_k (see section 1.2), it is possible to proceed to the assessment of vulnerability and fragility curves.

Fragility curves may be defined by lognormal functions (e.g. as proposed in HAZUS [HAZ 99]) that describe the probability of reaching, or exceeding, a defined damage state, given deterministic (median) estimates of spectral response (e.g. spectral displacement). In particular, the conditional probability $P\,[D_k \,|\, d_{PP}]$ of being in, or exceeding, a particular damage state (D_k), given the spectral displacement at the performance point d_{PP}, is defined by the following expression:

$$P\left[D_k \,\middle|\, d_{PP}\right] = \Phi\left[\frac{1}{\beta}\ln\left(\frac{d_{PP}}{d_k}\right)\right] \quad [1.4]$$

where Φ is the standard normal cumulative distribution function and β is the normalized standard deviation of the natural logarithm of the displacement threshold d_k. By knowing the form of the spectrum, and therefore the relationship between spectral displacements and other parameters which may characterize the demand (such as the peak ground acceleration, denoted a_g), it is possible to represent fragility curves by properly changing the abscissa axis ($d_k \rightarrow a_{g,k}$; $d_{PP} \rightarrow a_{g,PP}$).

The β parameter aims to summarize the variability and uncertainties associated with different factors, such as capacity curve properties, damage levels, model errors and ground shaking. It may be estimated with different degrees of accuracy: on the basis of expert judgment or conventional values proposed in the literature (such as those proposed in HAZUS [HAZ 99]); as a function of some mechanical parameter aimed at summarizing the seismic response of the examined class (such as the ductility of the capacity curve as proposed in Lagomarsino and Giovinazzi [LAG 06b]); and from a more refined probabilistic assessment (e.g. as proposed in Pagnini *et al.* [PAG 11]).

Figure 1.10 summarizes the evaluation of a fragility curve according to results from mechanical models.

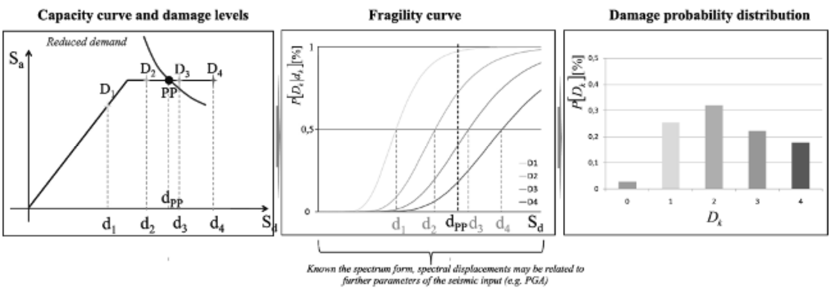


Figure 1.10. *Derivation of fragility curves from mechanical approaches*

1.4.1. *Masonry buildings*

Among the different mechanical models proposed in the literature (e.g. [CAL 99] and [RES 04]), in the following we describe the DBV-*masonry* method proposed in Lagomarsino *et al.* [LAG 10]. This model starts from the model that was originally proposed by Cattari *et al.* [CAT 05] with some modifications discussed in Pagnini *et al.* [PAG 08] and in Cattari *et al.* [CAT 10]. Moreover, this model has been recently implemented by Pagnini *et al.* [PAG 11] by including also a proposal for the probabilistic assessment. It is worth outlining that this model considers only the global response associated with the main activation of in plane response of masonry panels; an exhaustive assessment should be integrated by also including the out-of-plane response. Other models proposed in the literature take into account the combination of two mechanisms by introducing corrective factors (such as that proposed in Restrepo-Vélez and Magenes [RES 04]) or providing evaluations at the scale of masonry walls related to both failure modes (D'Ayala and Speranza [DAY 03]).

In the case of the model assumed as the reference, an analytical description of the capacity curve is provided as a function of a few geometrical, mechanical, technological parameters (number of floors, material strength, drift capacity, resistant area, etc.) and of a certain global collapse mode. In particular, the occurrence of two global mechanisms is considered: the soft-story and "uniform". The "uniform" mechanism indicates a collapse with a first localization of the damage on spandrels and with the subsequent collapse of piers only in the final phase. With respect to the original formulation, corrective factors have been introduced ([CAT 10, LAG 10]) in order to take into account some peculiarities of existing buildings and to improve the evaluation of the vibration period (e.g. to consider the flexural contribution to the stiffness, the coupling effect on masonry piers due to spandrel elements and the irregularities on the plan configuration).

By assuming no hardening, the capacity curve is defined by the three following entities (Figure 1.11): the yield acceleration a_y (basically related to the shear strength offered by the resistant wall area at ground floor); the fundamental period of the structure T_y (derived considering a linear mode shape and a structural stiffness related to the sole shear component); the ultimate displacement capacity d_u (related to drift limit values of masonry panels according to the failure mode considered and corresponding to d_4).

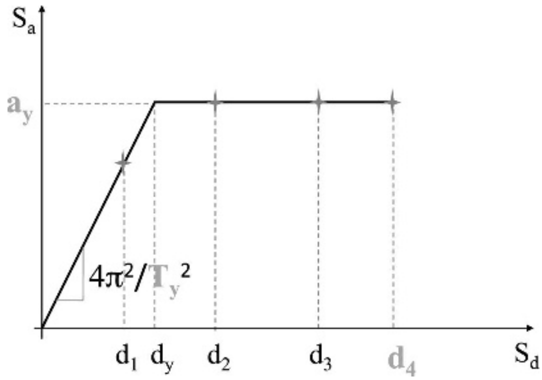


Figure 1.11. C*apacity curve assumed for the masonry building class (the entities directly computed on mechanical basis which univocally define the capacity curve are marked in gray)*

The equivalence in terms of SDOF (to properly compare the capacity curve with the demand in spectral form) is established by referring to the procedure proposed by Fajfar [FAJ 00] and also assumed as a reference in Eurocode 8 – Part 1 [EUR 05a], and thus by introducing the coefficient Γ and the equivalent mass *m**.

The assessment of the capacity curve is associated with a certain analysis direction (*dir* = X, Y), by then assuming the minor one as a reference in the case of scenario analyses.

The yielding acceleration $a_{y,dir}$ (for each examined direction) is provided as follows:

$$a_{y,dir} = \frac{F_{dir}}{m^* \, \Gamma} \qquad [1.5]$$

where F_{dir} is the total base shear capacity. The coefficient Γ requires the assumption of a modal shape ϕ:

$$\Gamma = \frac{\sum m_i \, \phi_i}{\sum m_i \, \phi_i^2} = \frac{m^*}{\sum m_i \, \phi_i^2} \qquad [1.6]$$

where m_i is the mass of the i-th story, m^* is the equivalent mass of the SDOF and ϕ_i is the component of the assumed modal shape ϕ in which the components are normalized in such a way that $\phi_N = 1$, with N being the top floor). It is worth noting that the same coefficient Γ is applied for the transformation of both displacements and forces. In particular, the assumption of a linear displacement shape is proposed to approximate the first mode shape. According to this assumption, the i-th component of ϕ is computed as:

$$\phi_i = \frac{z_i}{H} \quad [1.7]$$

where z_i is the altitude of the i-th story and H is the total height. In particular, even varying the supposed collapse mode (if uniform or soft-story), the same shape for ϕ is assumed; in fact, no significant modification in deformation is expected in the elastic range between these two different collapse mechanisms.

The total base shear capacity F_{dir} is basically related to the shear strength offered by the resistant walls area at the first floor level ($A_{1,\mathrm{dir}}$); only the contribution of walls parallel to the examined direction is considered. In particular, it may be computed as follows:

$$F_{dir} = A_{1,dir}\, \tau_{u,dir}\, \xi\, \zeta_{res} \quad [1.8]$$

where $\tau_{u,dir}$ is the ultimate shear strength of the masonry; ξ is a coefficient aimed at penalizing the strength as a function of the main prevailing failure mode expected at the scale of masonry piers (assumed to be 0.8 in the case of flexural behavior prevailing and 1 otherwise); ζ_{res} is a corrective factor aimed at considering some peculiarities of existing building and irregularity effects as described in the following. With reference to $\tau_{u,dir}$, the ultimate shear is computed according to the criterion proposed by Turnsek and Cacovic [TUR 71] as:

$$\tau_u = \tau_k \sqrt{1 + \frac{\sigma_{0,dir}}{1.5\tau_k}} \qquad [1.9]$$

where τ_k is the shear strength and $\sigma_{0,dir}$ is the average vertical compressive stress at the middle height of the first-level masonry piers. To compute $\sigma_{0,dir}$ it is necessary to consider the contribution of all acting loads on the resistant walls in the examined direction; it may be computed as:

$$\sigma_{0,dir} = \frac{g\left[\gamma \frac{h_1}{2} A_{1,dir} + \gamma \sum_{i=2}^{N} h_i A_{i,dir} + \sum_{i=1}^{N} A\, q_i\, \delta_{dir}\right]}{A_{1,dir}} \qquad [1.10]$$

where g is the gravity acceleration (equal to 9.81 m/s^2); h_1 and h_i are the inter-story heights of the first and i-*th* story, respectively; A_i is the resistant area at the i-th level; q_i is the load including the contribution of both gravity loads and variable actions (properly combined as proposed by codes in the case of seismic analysis); δ_{dir} (variable from 0 to 1) is a coefficient aimed at considering the main loading direction of the floors with respect to masonry walls.

The resistant area may also be expressed by introducing the α_{dir} and $\beta_{i,dir}$ factors, defined as follows:

$$\alpha_{dir} = \frac{A_{N,dir}}{A}; \quad \beta_{i,dir} = \frac{A_{i,dir}}{A_{N,dir}} \qquad [1.11]$$

where A is the total floor area; $A_{i,dir}$ and $A_{N,dir}$ are the resistant wall area at the i-th and top floor (N) level, respectively, in the examined direction. The $\beta_{i,dir}$ factor aims to characterize changes of resistant wall area in height. The introduction of these factors allows us to remove the explicit dependence on A in the above introduced expressions (as discussed in Cattari *et al.* [CAT 05] and Pagnini *et al.* [PAG 08]).

The evaluation of period ($T_{y,dir}$) is based on the proposal of Pagnini *et al.* [PAG 08], which mainly refers to the contribution of the shear stiffness. From the general definition of the period of the SDOF system it follows:

$$T_{y,dir} = 2\pi \sqrt{\frac{m^*}{k^*_{dir}}} \qquad [1.12]$$

where k^*_{dir} is the stiffness of the SDOF system and m^* is computed as introduced above as a function of a linear mode shape. It is worth noting that, since the bilinear behavior assumed for the capacity curve is an approximation of the actual response, this period has to be considered as representative of a partially cracked state; thus, it does not represent the initial period associated with the fully elastic condition (as a consequence usually mechanical parameters representative of cracked conditions are assumed). In particular, k^*_{dir} is computed from the following expression:

$$k^*_{dir} = \zeta_{rig} \frac{G}{H^2} \sum_{i=1}^{N} A_{i,dir} h_i \qquad [1.13]$$

where G is the shear modulus of masonry. Starting from the original proposal of Pagnini *et al.* [PAG 08], a corrective factor ζ_{rig} has been introduced, which aims to summarize the effects related to the coupling effectiveness of spandrels and to the flexural contribution; further details on ζ_{rig} are provided in the following paragraphs.

Some of the above-mentioned expressions may be further simplified, for example by defining a constant value for the inter-story height or by assuming a certain distribution of the resistant area (i.e. by defining a fixed function for $\beta_{i,dir}$). These assumptions allow us to analytically define in close form the expressions for $a_{y,dir}$ and $T_{y,dir}$ as proposed in Pagnini *et al.* [PAG 08].

Regarding the corrective factors ζ_{rig} and ζ_{res}, their introduction is aimed at considering some of the features that often characterize existing buildings.

As an example, it is worth noting that the evaluation of the strength capacity (equation [1.7]) implicitly assumes that all masonry piers fail at the same time, that is by supposing them fully coupled. This assumption is more consistent to the shear-type frame model, which is usually associated with the occurrence of soft-story failure. However, in the case of existing buildings, this hypothesis is far from being verified leading in many cases to "uniform" or "mixed" mechanisms. Figure 1.12 summarizes the effects of the coupling effectiveness of masonry piers on both the terms of deformed shape at collapse and distribution of the generalized forces (shear and bending moment) of a masonry building subjected to seismic load, passing from the case of very weak spandrels (case *a*) to the shear-type idealization (case *c*). Usually, the presence of specific constructive details plays a further crucial role in addressing the choice between the two extreme idealizations (*a*) and (*c*). For that matter, in general, case (*c*) seems consistent with new buildings in which masonry spandrels are always connected to lintels, tie beams and slabs made up of steel or reinforced concrete. In fact, these elements, being stiff and tensile resistant, assure a consistent coupling between piers, making the contribution of masonry negligible. On the contrary, in historical and existing buildings, spandrels are in many cases intrinsically weak elements. In fact, lintels are usually made up of wood or masonry, tie beams are often absent and floors are flexible (e.g. due to the presence of vaults or wooden floors): thus case (*a*) or (*b*) seems much more representative.

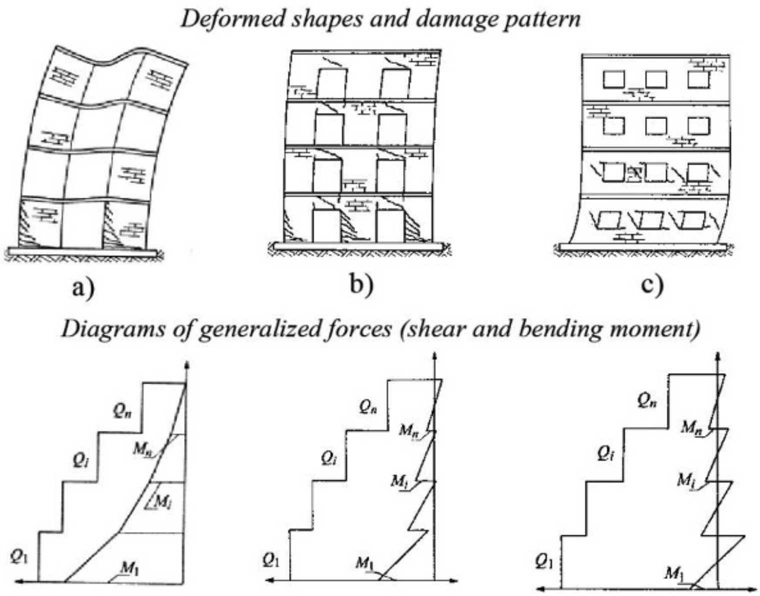


Figure 1.12. *Effects of effectiveness of spandrel coupling on masonry pier and global response: from the case of vey "weak" spandrels a) to the shear type idealization c); case b) represents an intermediate condition [TOM 99]*

Figure 1.13 summarizes, in the case of a three stories masonry wall subjected to seismic load (uniform load pattern), the potential effect of the coupling effectiveness of masonry piers not only on the overall shear strength but also in terms of both stiffness and displacement capacity. Cases (a) and (c) (which refers to the conditions illustrated in Figure 1.12) define the range of the possible pushover curves that can be associated with the structure.

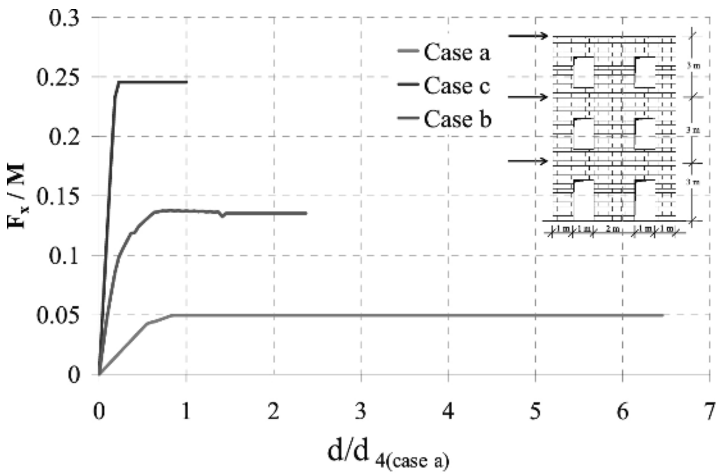


Figure 1.13. *Sensitivity of global response as a function of hypotheses assumed for the coupling of masonry piers (where M is the total mass of masonry wall)*

In order to take into account this and other effects (e.g. related to plan/elevation irregularities) and to obtain a more reliable evaluation of F_{dir}, the ζ_{res} factor combines these contributions:

$$\zeta_{res} = \zeta_1 \zeta_2 \zeta_3 \qquad [1.14]$$

where ζ_1 takes into account the influence of the non-homogeneous size of the masonry piers; ζ_2 the influence of geometric and shape irregularities in the plan configuration; and ζ_3 the influence of the global failure mechanism of the building (if soft-story or uniform), as a function of effectiveness of coupling spandrels. An analytical formulation for ζ_1 and ζ_2 may be found in the document of recommendations issued by the Italian Ministry of Cultural Heritage Assets [DIR 11]; however, since these formulations imply a degree of accuracy of available data incompatible in most of the cases with aims of vulnerability analyses at the territorial scale, some reference ranges are proposed in Table 1.3.

Corrective factor	Uniform failure mode	Soft-story failure mode
ζ_1	from 0.8 to 1	
ζ_2	from 0.75 to 1	
ζ_3	from 0.6 to 1	1

Table 1.3. *Corrective factors proposed for strength evaluation*

Similarly, as regards ζ_{rig} factor, it accounts for different contributions that may be summarized as follows:

$$\zeta_{rig} = \zeta_4 \zeta_5 \qquad [1.15]$$

where ζ_4 is a coefficient aimed at taking into account the influence of the flexural component on the stiffness and ζ_5 is a coefficient that considers the influence of the spandrels on the boundary conditions on piers. As regards ζ_4, if detailed geometrical data on the length of each masonry pier are available, the influence of the flexural component should be evaluated as follows:

$$\zeta_4 = \frac{1}{1 + \frac{1}{1.2}\frac{G}{E}\left(\frac{h_p}{b_p}\right)^2} \qquad [1.16]$$

being h_p and b_p the height and width of masonry piers, respectively; G and E the shear and Young's modulus, respectively. This contribution is rigorous only in the case of a single pier; in the case of an evaluation at territorial scale, the values of h_p and b_p should be intended as representative of the mean value for the entire stock examined. However, operating at the territorial scale it would be difficult to define these parameters (i.e. a mean value of the pier slenderness and a mean value of the ratio between G and E). Thus, as an alternative, in an approximate way, a range of variation from 0.4 to 0.8 is proposed in Table 1.4; within this range, the value of ζ_4 may be assigned as a function of the percentage of openings present in the masonry walls.

Corrective factor	Uniform failure mode	Soft-story failure mode
ζ_4	from 0.4 to 0.8	
ζ_5	from 0.7 to 1	1

Table 1.4. *Corrective factors proposed for stiffness evaluation*

As regards ζ_5, the characteristics of spandrels significantly affect the boundary conditions of piers (the effectiveness of the coupling among piers vary from the limit case of fixed-fixed end-rotation condition to that of a cantilever); this has a great influence on the prediction of their load-bearing capacity and, consequently, on the global response of the wall (see Figure 1.13).

The ranges proposed for ζ_i (i = 1, …, 5) have been calibrated on the basis of the comparison with results [CAT 10] carried out by detailed numerical nonlinear static analyses by using the Tremuri Program (which has been originally developed at the University of Genoa, starting in 2002 [GAL 09], and then implemented in the software 3Muri).

Finally, the ultimate displacement capacity d_u (corresponding to damage level 4) may be calculated as a function of the supposed collapse mode.

In the case of a uniform collapse mode, by assuming a linear deformed shape at collapse, d_u may be computed as:

$$d_{u,\ uniform\ mode\ (dir)} = \delta_{u,\ un\ (dir)} \frac{N h}{\Gamma} \qquad [1.17]$$

being $\delta_{u,un\ (dir)}$ the ultimate drift of masonry piers (the subscript *un* characterizes the prevailing failure mode in the pier associated with the uniform collapse mode).

In the case of soft-story collapse mode, the expression of d_u becomes:

$$d_{u,\ soft\text{-}story\ mode\ (dir)} = \delta_{u,\ ss\ (dir)}\ h + d_{y,dir}\left(1 - \frac{\Gamma}{N}\right) \qquad [1.18]$$

where the subscript *ss* means that the prevailing failure mode in piers is associated with the soft-story collapse mode and d_y is the yielding displacement (that may be computed starting from a_y and T_y as $(T_y/2\pi)^2 a_y$). Of course the occurrence of different failure modes depends on several parameters: the geometry of the piers, the acting axial load, the mechanical characteristics of the masonry and the masonry geometrical characteristics. In particular, the boundary conditions play a crucial role: thus it seems reasonable in assigning different drift values to the pier as a function of the main global failure that occurred in the building; if uniform or soft-story, since in these two cases, due to the coupling effectiveness of spandrels, the boundary conditions of piers may be different.

Of course, equations [1.17] and [1.18] are representative of two extreme conditions, thus it seems reasonable that the actual response of a masonry building is intermediate between these two displacement capacities. In particular, the ultimate displacement capacity d_4 should be defined as a proper combination of them:

$$d_4 = d_{u,soft\text{-}story\,mode\,(dir)} + (1-\varepsilon)d_{u,uniform\ mode\,(dir)} \quad [1.19]$$

ε being the fraction assigned to the soft-story global failure mode.

Once the capacity curve has been evaluated, the displacements values relating to the different damage levels have to be defined. In particular, the average values of the displacements threshold d_i (i = 1, 2) are proposed as a function of the yielding d_y; as a consequence, they may be expressed as analytical functions of the mechanical and geometrical parameters on which the model is based. Considering that the period is associated with a cracked state, it seems coherent to define the slight damage (d_1) before the yielding displacement d_y. On the other hand, a moderate damage (d_2), corresponding to the achievement of the maximum strength, is expected to be attained for a spectral displacement greater than d_y. In particular, the following relationships (based on expert judgment) are assumed:

$$d_1 = 0.7 d_y$$
$$d_2 = \rho_2 d_y \quad [1.20]$$

where ρ_2 is a coefficient that varies as a function of the prevailing global failure mode. In particular, the assumption of a value equal to 1.5 is proposed in the case of the soft-story failure mode and 2 in the case of the uniform failure mode. This differentiation is based on the different global behavior that occurs for these two failure modes. In particular, as previously introduced, in the case of uniform collapse mode, damage spreads progressively with an initial localization of the damage on spandrels and with a subsequent collapse of piers only in the final phase: thus the pushover curve is strongly nonlinear just from the beginning. On the contrary, in the case of soft-story collapse, damage in piers occurs suddenly: this justifies the definition of the damage limit state 2 closer to d_y than in the case of a uniform mode. Moreover, in the case of the soft-story failure mode, since damage in piers strongly compromises both the operativeness and repairability of the building, it seems justifiable that the distance of the slight damage state (d_1) from the moderate state (d_2) is smaller than in the uniform case.

Finally, with reference to d_3, it seems reasonable to define it by assuming a formulation analogous to that of d_4, that is from equation [1.19], by properly defining ranges of drift of masonry panels ($\delta_{3,un\ (dir)}$) aimed at graduating the damage level. Both national and international codes propose drift limit values as a function of the main failure mode occurring in the panel. As an example, Eurocode 8 – part 3 [EUR 05b] and the Italian Technical Code [ITA 08] propose values equal to 0.004 and 0.008 (which may be reduced to 0.006 in the case of existing buildings) for the shear and rocking failure modes, respectively. Actually, these limit values seem much more representative of a damage level 3, being on the safe side, because they are used for the design (in this case they are used for the assessment). According to this, it seems reasonable to assume higher drift limits in the case of a damage level 4 (a value of 0.01 seems acceptable).

1.4.2. *Reinforced concrete buildings*

Among various models proposed in the literature for RC buildings (e.g. in [DOL 04] and [CAL 99]), the DBV-*concrete* method proposed in Lagomarsino *et al.* [LAG 10] and in Cattari *et al.* [CAT 12a] is discussed in the following. This model basically starts from the model originally proposed by Crowley *et al.* [CRO 04, CRO 08], called DBELA, with some modifications introduced by the authors mainly related to the definition of the yielding period (by the introduction of the ψ coefficient) and the SDOF conversion (by the introduction of the κ' coefficient).

Actually, the DBELA method is derived from the direct displacement-based design method [CAL 99] and its application does not strictly require the outline of the capacity curve; however, all variables necessary to define it are implicitly introduced. In particular, the capacity curve (by assuming a bilinear curve without hardening) is defined through the vibration period (T_2) the displacement capacity at yielding (d_2) and the ultimate displacement (d_4); the expressions of T_2, d_2 and d_4 are differentiated as a function of various structural types and two main global failure modes (beam-sway or colum-sway). Once d_2 and T_2 are defined, the ultimate strength of the capacity curve (a_y) is obtained through their intersection ($a_y = d_2(2\pi/T_2)^2$). Displacement capacities are basically related to the chord rotation of the main structural element, column or beam. Depending on the global failure mode the evaluation of the fundamental period T, which in the original formulation was basically related to the building height, has been modified by the authors taking into account some additional mechanical parameters that may influence the response. Originally in the DBELA method, an elastic perfectly plastic behavior is assumed by considering only three limit states, starting from D_2. In order to also define the first damage state (D_1), associated with the non-structural light damage condition, the capacity curve could be modified through the appropriate principles; for example, it has been modified by the authors as shown in Figure 1.14 by defining the elastic period T_1 and relating d_1 to a proper percentage (ζ) of the overall strength. Further details related to the definition of the D_1 are shown in the following.

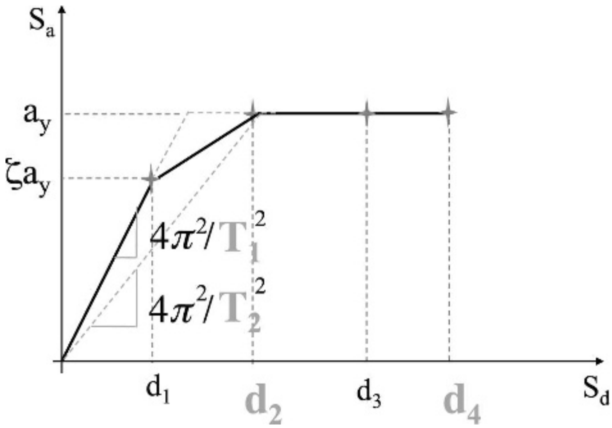


Figure 1.14. *Capacity curve in case of RC building class (the entities directly computed on mechanical basis which univocally define the capacity curve are marked in gray)*

The equivalence in terms of SDOF is established by introducing an effective height coefficient (κ_1), defined as the ratio between the height of the center mass of the SDOF substitute structure (H_{SDOF}) and the total height of the original structure (H_T) (as shown in Figure 1.15(a)).

Priestley [PRI 97] proposed to define κ_1 considering that the center of mass of the SDOF has the same displacement capacity of the original structure at its center of seismic forces (H_{CSF}). The coefficient κ_1 is a function of deformed shape and, through that, also a function of the prevailing failure mode (if column-sway or beam-sway), level of ductility and building height. As introduced by Priestley [PRI 97], Figure 1.15(b) shows three possible displaced shapes of a frame structure with the same rotation θ at the base subjected to an inverted triangle pattern load. In particular, the linear profile 1 corresponds to both: the elastic and inelastic deformed shapes of beam-sway frames of four or fewer stories and the elastic deformed shape of column-sway frames of any height. Profile 2, assumed as parabolic, shows elastic and inelastic deformed shapes of beam-sway frames until 20 storys; in fact, as noted in Priestley [PRI 97], dynamic inelastic analyses indicated that at peak response the plastic displacement profile for these structures is nonlinear, with larger plastic drifts occurring in the lower floors. Finally, profile 3 represents the inelastic deformed shape of a column-sway frame. Starting on these basic deformed shapes, the height of the center of seismic force (H_{CSF}) can be estimated; however, as stressed by Priestley [PRI 97], it must be recognized that the center of seismic forces depends on the displaced shape. It means that if an inverted triangle shape is a reasonable approximation of the elastic displacement response, the inelastic displacement increases as the center of the seismic force gradually decreases.

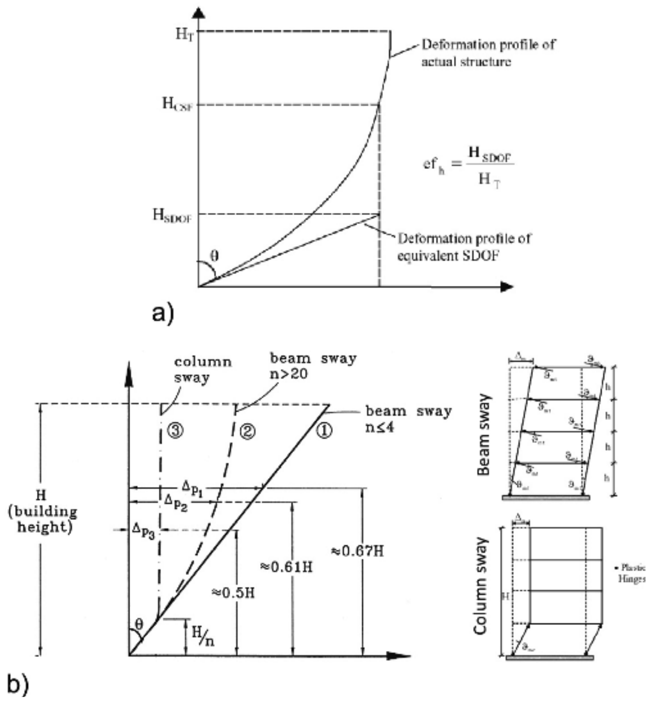


Figure 1.15. *a) Definition of effective height coefficient (from Glaister and Pinho [GLA 03]); b) deformation profiles as a function of the failure mode, building height and ductility (from Priestley [PRI 97])*

On the basis of the previous considerations, as proposed in Priestley [PRI 97] and also assumed in Crowley *et al.* [CRO 04], the κ_1 coefficient may be computed as summarized in Table 1.5 as a function of different damage states and global failure modes. It is worth noting that in the case of a column-sway frame the different expressions reflect the effect due to the variation of the center seismic force, depending on the progression of inelastic displacements; on the contrary, this effect may be neglected in the case of beam-sway frame.

<table>
<tr><th rowspan="2">Damage level</th><th colspan="2">Global failure mode</th></tr>
<tr><th>Column-sway</th><th>Beam-sway</th></tr>
<tr><td>D_1</td><td rowspan="2">$\kappa_1 = 0.67$</td><td rowspan="4">$\kappa_1 = \begin{cases} 0.64 & N \leq 4 \\ 0.64 - 0.0125(N-4) & 4 < N < 20 \\ 0.44 & N \geq 20 \end{cases}$</td></tr>
<tr><td>D_2</td></tr>
<tr><td>D_3</td><td rowspan="2">From Priestley [PRI 97]:
$\kappa_1 = 0.67 - 0.17\frac{\mu_i - 1}{\mu_i}$
or
From Glaister and Pinho [GLA 03]:
$\kappa_1 = 0.67 - 0.17\frac{\varepsilon_{Si} - \varepsilon_y}{\varepsilon_{Si}}$</td></tr>
<tr><td>D_4</td></tr>
<tr><td colspan="3">with N total number of storys; μ_i ductility associated with the i-th damage level examined (computed starting from D_2); ε_{Si} and ε_y steel strain corresponding to the i-th damage level and that to the yielding, respectively.</td></tr>
</table>

Table 1.5. *Evaluation of κ_1 coefficient*

With respect to the original proposal of Priestley [PRI 97], the DBV-*concrete* method has introduced some modifications to the expressions proposed in Table 1.5, which are particularly relevant in the case of low-rise buildings. Actually, in the case of the inverted triangle load pattern, the center of seismic forces is located at 0.67 H_T (i.e. 2/3 of H_T) only in the case of a continuum system. In fact, in the case of a building characterized by a single level, by concentrating all the seismic force at the top, the center of seismic mass is located at H_T. If the Γ coefficient (as used in the case of masonry structures and illustrated in section 4.2) is introduced, by supposing that all the masses m_i are equal and by assuming a linear modal shape ($\psi_i = i/N$), the equation [1.5] becomes:

$$\Gamma = N\frac{\sum i}{\sum i^2} = \frac{3N}{2N+1} \quad [1.21]$$

that, in the case of N = 1, provides the value equal to 1. As previously introduced, the Γ coefficient is applied to both displacements and forces to establish the equivalence with the SDOF. On the basis of the previous issues, it seems reasonable to apply a corrective coefficient to κ_1 aimed at taking into account the mismatch noted in the position of the center of seismic mass in the case of buildings characterized by a few stories and with masses prevailing concentrated at floor level. In particular, this corrective factor is aimed at scaling the κ_1 value as a function of the limit condition provided by equation [1.21]. In particular, it is proposed to multiply κ_1 for the following corrective factor:

$$k' = \frac{2N+1}{2N} \quad [1.22]$$

Regarding the definition of the elastic period corresponding to D_1, according to the formula proposed in

Eurocode 8 – Part 1 [EUR 05a], the following relationship is adopted:

$$T_1 = C_1 H_T^{\beta_1} \qquad [1.23]$$

where C_1 and β_1 coefficients aim to take into account the structural type of the RC building (bare frames, infilled frames, dual system, etc.). As an example, in the case of reinforced concrete moment resisting frames (in which the interaction with masonry infill panels is not significant), Eurocode 8 proposed values equal to 0.075 and 0.75 for C_1 and β_1, respectively. It is worth highlighting that these values are representative of the assumption of gross stiffness section properties: hypothesis coherent for the elastic range in which a low damage is expected. A further distinction should be made as a function not only of the structural type but also of the level of a seismic design. As an example, principles of the capacity design force the occurrence of a beam-sway failure mode rather than the column-sway; on the contrary, a design that takes into account only gravity loads (pre-code situation) usually produces smaller (and thus more flexible) column sections. Pinho and Crowley [PIN 09] show that the formula proposed in Eurocode 8 for moment resisting frames, matches well the period of vibration of newer European buildings (post-1980 frames). Figure 1.16(a) shows the comparison between the numerical results obtained by Crowley [CRO 03] on the bare frame representative of two different design codes (with or without seismic details) and some empirical formulas proposed in literature (in particular, those proposed in Goel and Chopra [GOE 97]) and in Eurocode 8 – Part 1 [EUR 05a]. In particular, it seems that the upper bound proposed by Goel and Chopra [GOE 97], which corresponds to assume $C_1 = 0.065$ and $\beta_1 = 0.9$, matches well the period of vibration of reinforced concrete moment resisting frames designed only for vertical loads without significant seismic details (pre-1980 frames).

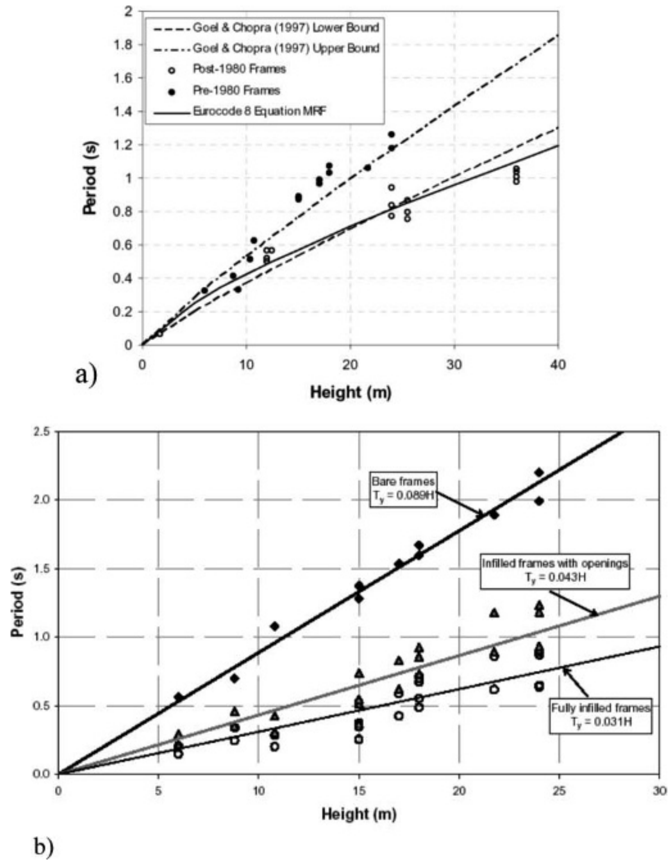


Figure 1.16. *a) Comparison between numerical results provided by Crowley [CRO 03] and some empirical formulas proposed in the literature and codes (from Pinho and Crowley [PIN 09]); b) analytical yield period-height relationship for cracked stiffness properties (from Crowley and Pinho [CRO 06])*

With reference to the definition of the period that corresponds to D_2, in the original proposal of DBELA [CRO 04], the period at yielding was computed as 0.1 H_T (being H_T the total height of the building). However, this assumption presents the following main drawbacks: (1) it does not take into account the dependence of the type of the RC structure (regarding this, a much more detailed characterization is proposed in Crowley and Pinho [CRO 06], in which the linear relationship is calibrated as a function of the type of RC structure, such as bare or infilled frames as illustrated in Figure 1.16(b); (2) it does not take into account the influence of other mechanical parameters (e.g. the size of the column influences not only the strength but also the stiffness). To overcome these drawbacks, the following relationship has been proposed by the authors [LAG 10]:

$$T_2=\psi\left(C_2 H_T^{\beta_2}\right)=\psi\bar{T}_2 \qquad [1.24]$$

where

– ψ represents a coefficient aimed at taking into account the dependence of the period on the variation of certain geometric and mechanical parameters. It derives from simple considerations on the parametric dependence of the period with these factors, as derived from modal analyses:

$$\psi=\left(\frac{\bar{h}_s}{h_{si}}\right)^{0.5}\left(\frac{\bar{h}_{sT}}{h_{sTi}}\right)^{\beta_3}\left(\frac{H_{Ti}}{\bar{H}_T}\right)^{0.75}\left(\frac{\bar{f}_c}{f_{ci}}\right)^{0.125} \qquad [1.25]$$

where

– $\bar{h}_s, \bar{h}_{sT}, \bar{H}_T, \bar{f}_c$ are the parameters that correspond to the mean behavior of the class as previously defined: they correspond to the height section of column and beam, the total height of the building and the compressive strength of concrete, respectively;

– $h_{si}, h_{sTi}, H_{Ti}, f_c$ are the current values assumed as representative of the buildings stock;

– β_3 is assumed to be equal to 0.25 in the case of column-sway mechanism and 0.5 in the case of the beam-sway; this difference is justified by the consideration that period T_2 is representative of a nonlinear phase of the response in which the different deformed shape of the structure, in the case of column- or beam-sway, also plays a role.

– $\overline{T}_2$ represents the reference value of the period for the examined class (obtained by adopting the values of C_2 and β_2 defined in the following). It means that for a fixed set of parameters, which are assumed to be representative of the mean behavior of the class (in particular, $\overline{h}_s, \overline{h}_{sT}, \overline{H}_T, \overline{f}_c$), the ψ coefficient is assumed to be equal to 1. Then, the period $\overline{T}_2$ is expressed by a relationship analogous to that of the elastic period (T_1); however, it is important pointing out that for the evaluation of $\overline{T}_2$ reference is made to the effective stiffness properties.

Regarding C_2 and β_2 coefficients, as an example, in the case of moment resistant frames, reasonable values to be adopted seem to be, respectively: (1) 0.089 and 0.9 in the case of frames designed according to recent seismic design codes (post-1980 frames); (2) 0.089 and 1 (as proposed in Crowley and Pinho [CRO 06]) in the case of frames designed only for vertical load without significant seismic details (pre-1980 frames).

With reference to the values of the period at damage level 3 and 4, they are related to T_2 through the corresponding value of the ductility μ_i as follows:

$$T_i = T_2 \sqrt{\mu_i} \qquad [1.26]$$

It is important to highlight that the ductility μ_i is computed starting from the capacity displacement corresponding to level 2 (as the ratio d_i / d_2).

The last important aspect of the model is the definition of the displacement capacity d_i for each damage level.

From D_2 to D_4, the displacement capacity is basically related to the chord rotation capacity of the main element that determines the response of the structure, that is the RC beam in the case of the beam-sway frame or the RC column in the case of column-sway frame. Actually, in the case of the beam-sway mechanism, it is assumed that beams and columns located at the base have the same rotation; in this case, although the main structural elements are assumed to be the beams, it seems important to check if the chord rotation, computed assuming as reference values parameters that characterize beams, is compatible with the maximum rotation that occurs in the columns.

In the case of d_2 the displacement capacity is related to the chord rotation (θ) corresponding to yielding ($\theta_y = \theta_2$); thus, d_2 can be evaluated as follows:

$$d_2 = \kappa_1 \theta_2 H_T \qquad [1.27]$$

In the original proposal of the DBELA method [CRO 04], the yield chord rotation (θ_2) is provided by two different formulas in order to distinguish the beam-sway failure mode from the column-sway failure mode. However, since these formulas contain some coefficients (calibrated on an experimental basis), a much more general formulation of the chord rotation (like that proposed in Panagiotakos and Fardis [PAN 01] and also in Eurocode 8) has been assumed. This formulation allows us to explicitly take into account the dependence on other mechanical parameters that govern the response (such as the resistance of steel and concrete, the diameters of bars, the transversal column section and the inter-story height). In particular, θ_2 may be computed from:

$$\theta_2 = \varphi_y \frac{L_v}{3} + 0.0013\left(1 + 1.5\frac{h_{s(T)}}{L_v}\right) + 0.13\varphi_y \frac{d_b f_y}{\sqrt{f_c}} \qquad [1.28]$$

where

– ϕ_y is the yield curvature of the section, which is calculated according to the relationships proposed by Priestley [PRI 97] as:

$$\phi_y = 2.14\frac{\varepsilon_y}{h_s} \quad \textit{in the case of column element}$$
$$\phi_y = 1.7\frac{\varepsilon_y}{h_{s,T}} \quad \textit{in the case of beam element} \qquad [1.29]$$

where

– ε_y is the yield strain of the longitudinal rebars of the element and $h_{s(T)}$ is the section height of the main structural element that governs the global response (if beam or column);

– L_v is the shear span (equal to the ratio between the bending moment and shear); usually, it is assumed to be half of the height element (i.e. half of the inter-story height in the case of a column element), assuming a double bending distribution;

– d_b is the longitudinal bar diameter;

– f_y and f_c are the strength of steel and concrete in MPa, respectively.

Finally, in the case of D_3 and D_4, the post-yield displacement capacity is obtained by adding a plastic component to the yield chord rotation. In the case of a column-sway mechanism, the plastic component is concentrated on the columns on the ground floor (assuming that the soft-story is always located at the base of building); in the case of beam-sway also in the post-elastic range, a linear shape is assumed. In particular, it is important to highlight that in the case of a beam-sway failure mode it is assumed that the entire height of the building is involved in the mechanism; actually, if known, it should be possible to take into account the occurrence of this mechanism starting from a certain level considering the center of mass moving toward the center of mass of the part of the building that is involved in the collapse (as proposed in Borzi *et al.* [BOR 08]). Thus the displacement capacity d_i (i = 3, 4) is defined by the following equations:

$$d_i = \kappa_1 \theta_2 H_T + (\theta_i - \theta_2)\kappa_1 H_T \quad i = 3,4 \text{ in the case of beam-sway mechanism}$$
$$d_i = \kappa_1 \theta_2 H_T + (\theta_i - \theta_2) h_1 \quad i = 3,4 \text{ in the case of column-sway mechanism} \qquad [1.30]$$

where h_1 is the inter-story height at ground floor.

The chord rotation at limit state i (θ_i), which is related to the ultimate rotation capacity, is computed as:

$$\theta_i = \alpha_i \frac{1}{\gamma_{el}} \left(\theta_2 + (\phi_u - \phi_y) L_{pl} \left(1 - \frac{0.5 L_{pl}}{L_v} \right) \right) \qquad [1.31]$$

where

– α_i is a coefficient aimed at limiting the value of the ultimate rotation capacity. It is usually assumed equal to 1 for D_4 and 0.75 for D_3, respectively (as proposed also in Eurocode 8). However, it seems reasonable to also assume different values if calibrations based on comparisons with survey damage or experimental data are available. As an example, in Lagomarsino *et al.* [LAG 10], on the basis of results carried out from the calibration of this mechanical model with the damage scenario that occurred in L'Aquila on April 6, 2008, it seems justifiable to assume α_3 equal to 0.6;

– γ_{el} is equal to 1.5 for primary structural elements and 1 for all others; usually, in the case of mechanical models, in which only the contribution of primary elements is taken in to account, it is assumed indistinctly equal to 1.5;

– ϕ_u is the ultimate curvature, which is assumed to be:

$$\phi_u = \frac{\varepsilon_{cu} - \varepsilon_{su}}{h_{s(T)}} \qquad [1.32]$$

where ε_{cu} and ε_{su} are the ultimate concrete and steel strains, respectively. Regarding this, ranges of values suggested by

Calvi [CAL 99], which could be assumed as the reference, are the following: in the case of poorly confined RC elements ε_{cu} = 0.5–1% and ε_{su} = 1.5–3%; in the case of well-confined RC elements, ε_{cu} = 1–2% and ε_{su} = 4–6%. Usually, in the case of existing buildings designed without specific seismic details (pre-code buildings) the condition of poor confinement has preferably to be assumed.

– L_{pl} is the plastic hinge length. It can be calculated as (according also to that proposed in Eurocode 8):

$$L_{pl}=0.1L_v+0.17h_s+0.24\frac{d_b f_y}{\sqrt{f_c}} \quad [1.33]$$

Regarding the formulas discussed above for θ_i (i = 2, 3, 4), it is worth outlining that many alternative formulations (both on mechanical or empirical basis) are proposed in the literature; thus the expressions introduced above could be replaced by relationships considered more reliable from the case examined. Regarding this, in Cattari *et al.* [CAT 12a], results of sensitivity analyses on the use of mechanical and empirical approaches to compute the chord rotation have been discussed paying particular attention to the repercussion on the vulnerability assessment.

Finally, with reference to the displacement capacity at limit state 1, it is assumed from the following expression:

$$d_1=\varsigma\, d_2\left(\frac{T_1}{T_2}\right)^2 \quad [1.34]$$

where the ζ coefficient is assumed to be equal to 0.9. In particular, in [1.34] the following assumptions have been made: (1) the ultimate strength of the capacity curve (a_y) is obtained through the intersection between d_2 and T_2; (2) the displacement capacity d_1 is fixed as the point of the curve in which a certain fraction ζ of A_y is reached. An alternative possible approach [CAL 99] could be to relate the displacement capacity d_1 to the inter-story drift capacity of the non-structural components, such as partition walls.

Once d_2 and T_2 are defined, the ultimate strength of the capacity curve (a_y), as previously introduced, is obtained through their intersection. It is important to stress that a_y is very sensitive to a reliable estimation of d_2 and T_2. Regarding this, with respect to the original proposal of the DBELA method, it seems particularly relevant in the DBV-*concrete* method with the introduction of a ψ coefficient that allows us to take explicitly into account the dependence on certain mechanical parameters not only for the displacement capacity but also for the period. Figure 1.17 shows some functional dependence of a_y on mechanical parameters that the DBV-*concrete* model is based (e.g. in the case of a prevailing column-sway mechanism); further details are illustrated in Cattari *et al*. [CAT 12a].

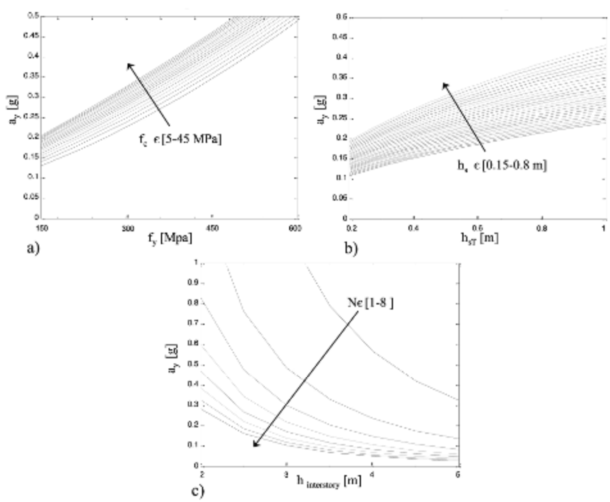


Figure 1.17. *Functional dependence of a_y on some mechanical parameters which the DBV-concrete model is based (case of column-sway failure mode) a) strength parameters (concrete compressive strength f_c and yielding steel strength* f_y*); b) structural element section's geometrical features (column – h_s – and beam – h_{sT} – height section); c) building geometrical features (story number N and inter-story height $h_{inter\text{-}story}$)*

Some recent applications of the DBV-*concrete* model are discussed in Cattari *et al.* [CAT 12a], Cattari and Ottonelli [CAT 12b], Cattari *et al.* [CAT 10] and Lagomarsino *et al.* [LAG 10]. In particular, the latter proposes a comparison of the simulated scenario with that which actually occurred in L'Aquila, after the earthquake on April 9, 2009 (with particular reference to the buildings in Pettino village and its surrounding area). The simulation has been conducted for different classes (as a function of the age, story number and structural type) characterized by homogeneous behavior to which is associated a proper mechanical model. Figure 1.18 shows the comparison between the simulated and real damage scenario as a function of the ages and storys number of examined classes. Despite the need of some improvements, the proposed methodology seems to provide a quite good and realistic assessment of the damage scenario that occurred. In fact, from the application, a percentage of not safe buildings equal to 27% against the surveyed scenario equal to 35% have been obtained.

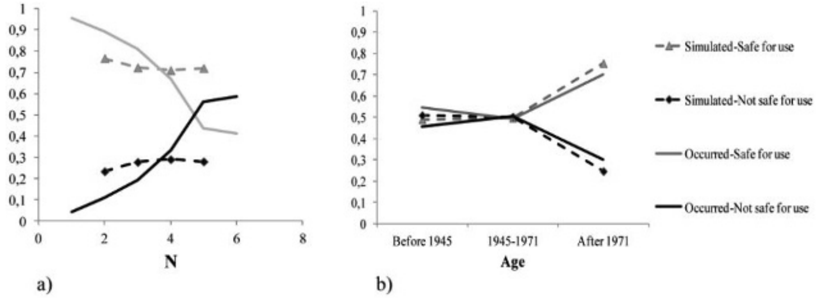


Figure 1.18. *Comparison between trends of simulated and surveyed scenario varying both ages and N*

1.5. Implementation of models for scenario analysis at territorial scale

In the previous sections, some *vulnerability models* have been discussed. Through them the response of a building stock characterized by homogeneous behavior may be analyzed: to this end, it is necessary to define at first, for each building class, the parameters of the model, which are different depending on the type of model (macroseismic and mechanical) and structural material (masonry and RC). These parameters can be derived by the statistical analysis of available data, when building surveys have been carried out; otherwise, some sample surveys can be made or, in the case of modern buildings, it is possible to get parameters from a simulated design, by considering codes of that time. Validation of the vulnerability models is possible by detailed analyses on prototype buildings.

For the implementation of the *macroseismic method*, described in section 1.3, the available data and information have to be properly processed in order to recognize the building type and, when possible, the relevant parameters able to affect the seismic behavior. As a matter of fact, an initial value of the vulnerability index V (referred to as typological vulnerability index V^*) may be attributed depending on the type of the vertical structures or further considering information about the class of height and the horizontal structure types. Moreover, for buildings designed according to a seismic code, the seismicity of the area and the ductility prescription can be considered.

The value of the typological vulnerability index V^* can be refined when, in the region or town, the evidence exists about a better or a worse performance with respect to the average performance, defined by the macroseismic method. This evidence can be obtained from the available observed damage data or from the judgment of local experts. The difference between the building type vulnerability index, specifically assumed for the region, and that proposed by the macroseismic method is called the regional vulnerability modifier ΔV_r.

Structural, technological and geometrical features (e.g. plan and vertical regularity, maintenance conditions and retrofitting interventions) that are expected to change the seismic behavior can further refine the definition of the vulnerability index. To this end, scores for the behavior modifier ΔV_m have been proposed [GIO 04], calibrated on the basis of observed damage data and the expert judgment from previously proposed scoring procedures, such as ATC 13 [ATC 87], Benedetti and Petrini [BEN 84] and UNDP/UNIDO [UND 85]. Some reference values are proposed in Table 1.6 [LAG 06a]; specific modifiers have been introduced to take into account the effects on masonry buildings in historical centers and in an aggregated context.

Parameter	ΔV_m
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)

Table 1.6. *Reference values for vulnerability scores ΔV_m of the main parameters*

According to the macroseimic approach, soil conditions are accounted for, as well, within the vulnerability index, being the hazard evaluated in terms of macroseimic intensity with reference to rigid soil conditions. As a matter of fact, the macroseimic intensity is not a mechanical parameter and, for this reason, it does not allow us to consider the site effects that could affect buildings in a different way, depending on their typology and class of height. To this end, considering the different geotechnical acceleration, multiplier factors f_{ag} for an equivalent PGA have been evaluated for each building typology, class of height and soil class (according to Eurocode 8 prescriptions). By assuming a proper I–a_g correlation as follows (where c_1 represents the PGA value a_g corresponding to the reference intensity I and c_2 measures the rate of the PGA a_g increase with intensity I):

$$a_g = c_1 c_2^{(I-5)} \tag{1.35}$$

where intensity increments ΔI may be translated in terms of a soil amplification modifier ΔV_s, according to equation [1.1] [GIO 04]:

$$\Delta V_s = \frac{\Delta I}{6.25} = \frac{\ln fa_g}{\ln c_2} \frac{1}{6.25} \tag{1.36}$$

The vulnerability index V is computed, combining the contribution of the typological vulnerability index V^* with the ones provided by the regional modifier ΔV_r, the behavior modifier ΔV_m and the soil amplification modifier ΔV_s.

When data are available on each single building (or building stock with homogeneous behavior), the vulnerability index is evaluated as the sum of the scores associated with each single modifier ΔV_m, ΔV_r and ΔV_s

$$V = V^* + \Delta V_m + \Delta V_r + \Delta V_s \tag{1.37}$$

The macroseismic method can also be implemented when the available data do not allow for a direct typological identification. In this case, subclasses of buildings have to be identified on the basis of more general information (such as land use patterns, a building's age and the building material) rather than by the typological information. Inferences have to be established between these subclasses and the building types (e.g. in masonry buildings built before

1919, 40% were of rubble stone typologies and 60% of old brick masonry buildings). The vulnerability indices to be attributed to the subclasses are obtained combing, according to the assumed inferences, the indices attributed to the building types.

In the case of the *mechanical method*, described in sections 1.4.1 and 1.4.2, more data are required. Table 1.7 summarizes the parameter set on which mechanical models are based. Since proposed models are also differentiated as a function of different supposed failure modes, parameters and coefficients need to be properly defined for each aspect of this combination. Since at large scale mechanical models are not applied to single buildings, model parameters have to be defined according to statistical evaluations.

	Masonry buildings	Reinforced concrete buildings
Geometrical features	N (story number); h (inter-story height); β (ratio between the resistant wall area at level i and the resistant wall area at top floor level); α (ratio between the resistant wall area at top floor level and the total floor area)	N (story number); h (inter-story height); h_1 (inter-story height at ground floor); h_s (height section of the main structural element ruling the global response, i.e. the RC beam or the RC column); d_b (longitudinal bar diameter)
Mechanical parameters and loads	τ (shear strength); G (shear modulus); γ (material density); q (floor load); δ_u (drift values of masonry piers)	ε_{cu} (ultimate concrete strain); ε_y (yielding steel strain); ε_{su} (ultimate steel strain); f_y(yielding steel strength); f_c(concrete resistance); L_V (shear span)
Corrective factors	ξ_1, ξ_2, ξ_3 (affect the evaluation of A_y); ξ_4, ξ_5 (affect the evaluation of T)	ψ (affect the evaluation of the period)

Table 1.7. *Building parameters for the mechanical model implementation*

1.6. Final remarks

The vulnerability analysis at the territorial scale is a fundamental tool to plan the mitigation strategies and optimize the use of funds for strengthening interventions.

In this chapter, different models are proposed that refer to two main approaches: macroseismic and mechanical. The first approach, based only on qualitative information (regularity, state of maintenance, etc.), presents the main advantage to guarantee a direct calibration with damage data collected after earthquakes, in terms of macroseismic intensity. The second approach, based on a limited number of geometrical and mechanical parameters, allows us to take explicitly into account the different parameters that influence the structural response and use an accurate description of the hazard (response spectrum).

A combined use of these two different approaches is possible and advisable, in order to exploit advantages of both and guarantee the reliability of the obtained damage scenario. Moreover, despite the apparent complexity of the proposed models (in particular, in the case of mechanical approaches), they may be easily implemented in the Geographic Information System (GIS) environment, usually adopted for seismic risk analyses at territorial scale.

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