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PERPETUATE guidelines for seismic performance-based assessment of cultural heritage masonry structures

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Abstract Ancient monumental masonry buildings are complex structures that were not based on an engineered design, underwent many transformations during their life and often present lack of connections among the structural elements. Earthquakes are the main cause of damage for ancient masonry structures and, in order to reduce their vulnerability with compatible and light interventions, it is necessary to have accurate models for the seismic analysis, able to simulate the nonlinear behavior of masonry, and a well defined performance-based assessment procedure, aimed to guarantee the acceptable level of risk for the occupants and for the conservation of the monument itself. The paper outlines the guidelines that were developed within the PERPETUATE European research project. The wide variety of architectural assets is classified and the related proper modeling strategies are identified; moreover, immovable artistic assets are considered in the assessment. A displacement-based approach is adopted, because these structures crack even for low intensity earthquakes and can survive severe ones only if they have a sufficient displacement capacity. Safety and conservation requirements are proposed by considering distinct sets of performance levels, related to use and safety of people, conservation of the building and of the artistic assets that might be present. Some indications on the seismic hazard assessment are provided, considering the distinctive features of some types of ancient structures. Within the fundamental knowledge phase, sensitivity analysis is proposed in order to address and optimize the in-situ investigation and to define proper confidence factors, aimed to consider epistemic and statistical uncertainties. Different modeling approaches and methods of analysis are considered, depending on the characteristics of the structure; both static pushover and incremental dynamic nonlinear analyses are considered. Related verification procedures are defined to evaluate the seismic intensity measure, and the corresponding return period, which is compatible with each performance level that must be fulfilled.

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1 Introduction

Damage assessment after recent earthquakes showed the high vulnerability of some types of historical structures, such as churches, palaces and towers (Oliveira 2003; Dogangun et al. 2008; Lagomarsino 2012; Cattari et al. 2014a; Sorrentino et al. 2013). Earthquakes also proved that strengthening interventions adopted in the last century are invasive and sometimes are not effective or even increase the vulnerability of these structures. Thus, proper methods of analysis and verification procedures are required for the seismic assessment and the design of interventions, with the aim of mitigating risk to cultural heritage.

The preservation of cultural heritage assets must guarantee their capacity of lasting over time against decay, natural hazards and accidental events, without losing, as much as possible, their authenticity. Moreover, there is a need to ensure the safety of occupants, related to the use of the building (private or public). To this end it is necessary to make reference to the principle of “minimum intervention”, under the constraint of an “acceptable safety level”, a concept that still represents an open issue for monumental buildings. Furthermore, besides the preservation of the architectural value of the building, also immovable artistic assets should be considered in the assessment, such as: frescoes, stucco-works, pinnacles, battlements, banisters, balconies etc.

The seismic assessment of existing buildings, irrespective of whether they are historical, is a complex task, basically for two different reasons: a) the difficulty of interpreting and modeling the seismic response, because they have been designed without provisions for earthquake resistance and, in the case of ancient masonry structures, by an empirical approach; b) the difficulty of acquiring as-built information on material parameters and structural details, due to their spatial variability in the buildings and the need of avoiding invasive investigations.

The best known international standards (Eurocode 8—Part 3 CEN 2005; ASCE/SEI 41–13 2014) adopt the Performance-Based Assessment (PBA), which considers different Performance Levels (PLs) that must be fulfilled in the occurrence of corresponding earthquake hazard levels (defined by the return period). The need to check the attainment of PLs that are close to structural collapse strongly recommends the use of static nonlinear models and displacement-based procedures for assessment, as it is not possible to rely on linear analyses using the behavior factor approach, since existing buildings are not capacity designed.

The relevant role of aleatory and epistemic uncertainties has suggested, quite recently, to implement the fully probabilistic SAC-FEMA approach (Cornell et al. 2002) within the static or dynamic nonlinear procedures (Fajfar and Dolšek 2012; Pinto and Franchin 2014).

The assessment of cultural heritage assets is treated in some guideline documents (ICOMOS 2005; ISO 13822 2010; CIB 335 2010), which are not specifically addressed to seismic vulnerability but consider all possible causes of damage and deterioration, with the aim of making a diagnosis and design a strengthening intervention. All these documents point out the complex configuration of this kind of structures, also due to the relevant transformations that have usually occurred over time, as well as the difficulty of adopting a proper modeling strategy. A common denominator among these recommendations is the importance of the qualitative approach, based on historical analysis, the accurate investigation of structural details and the interpretation of seismic behavior, on the basis of observed damage in the building (due to previous events, if any) or on similar structures. ICOMOS (2005) proposes the qualitative approach as an alternative to the quantitative one (based on the use of structural modeling); pros and cons of the two methods are clarified, but it is evident that a combined use would be the optimal solution. ISO 13822 (2010) contains an informative annex on heritage structures, which suggests to perform a preliminary assessment, based on investigations and historical analysis, and to adopt a detailed assessment only when the qualitative approach is not sufficient to obtain a clear picture of the building safety (diagnosis); therefore numerical modeling is required only in the latter case, and the unavoidable model deficiencies are supplemented by qualitative information (historical or comparative approach, engineering judgment). The assessment procedure proposed by CIB 335 (2010) coincides substantially with that of ISO 13822 (2010), but in addition some specific proposals on how to define safety coefficients and the confidence factor are introduced.

It is worth noting that a preliminary assessment is usually sufficient for the diagnosis in many critical situations, such as material deterioration or soil settlement. On the contrary, the evaluation of seismic vulnerability without the support of calculations is overambitious, because the qualitative approach can only suggest the expected seismic behavior and the historical analysis is not sufficient to prove the building safety. The Italian Guidelines for the seismic assessment of cultural heritage (Recommendations P.C.M. 2011) clearly state it is not possible to avoid a quantitative calculation of the structural safety, by using models that are based on an accurate knowledge; eventually, the results can be slightly adjusted by taking into account qualitative evaluations.

Several research projects, in particular funded by the European Commission, were aimed to the preservation of historical structures. RISK-UE developed a procedure for the assessment at urban level of direct and indirect losses, following an earthquake scenario; the preservation of cultural heritage was treated at territorial scale, with simplified approaches aimed to achieve a priority list for planning mitigation strategies (Lagomarsino 2006). EU-CHIC (Žarnić et al. 2012) has created an identity card for tangible cultural heritage assets, buildings and monuments (classified according to Lagomarsino et al. 2011), with the aim of tracking environmental changes and human interventions. ONSITEFORMASONRY (Maierhofer and Kopp 2006) provided a comprehensive set of recommendations for the application of different test methodologies addressed to the evaluation of the state of the structure and material, useful for supporting the preliminary diagnosis and the as-built information process. With reference to seismic preservation, PROHITECH (Mazzolani 2009) and NIKER (Modena et al. 2013) developed and analyzed the effectiveness of different strengthening interventions, by experimental tests and the application to case studies. However, the proposal of an assessment procedure repeatable and verifiable, which leads to the quantitative evaluation of safety levels taking into account properly historical and qualitative information, was still missing.

To this aim, the PERPETUATE European research project (Lagomarsino et al. 2010) has developed guidelines that are coherent with the afore cited recommendations but frame the problem of the seismic assessment of cultural heritage assets and design of interventions within the PBA approach outlined by the international standards for current buildings (CEN 2005, ASCE/SEI 41–13 2014).

This paper illustrates the different steps of the proposed procedure. In particular: 1) proper nonlinear modeling strategies are proposed for different classes of historical buildings; 2) safety and conservation requirements are defined, in order to take into consideration also the cultural value of the building and the contained artistic assets; 3) investigations and surveys are planned on the basis of sensitivity analysis, which allows to avoid invasive tests, if they are not really useful, and to evaluate reliable confidence factors; 4) static and dynamic nonlinear verification procedures are defined, depending on the possibility of considering a global seismic behavior of the structure or the response of independent macroelements.

2 Seismic performance-based assessment of architectural and artistic assets

Seismic Performance-Based Assessment (PBA) of an existing building checks if the structure is able to fulfill some selected Performance Levels (PLs) in case of occurrence of properly defined earthquake hazard levels, in terms of annual rate of exceedance λ (or return period $T_R \approx 1/\lambda$).

Target PLs are properly defined in these guidelines for cultural heritage assets, which consider not only the use and safety of people but also the conservation of the architectural and artistic value of the monument.

Figure 1 summarizes the basic principles and steps of PBA according to PERPETUATE guidelines. Since pushover analysis is considered the standard tool for the PBA, detailed acceptance criteria are proposed for the identification of target PLs on the pushover curve, by considering the displacement u (or d once the MDOF structural response is converted into the equivalent SDOF) as Engineering Demand Parameter (EDP) and defining proper thresholds. When nonlinear dynamic analyses are used, IDA curve is represented in terms of the selected EDP, in order to check the attainment of thresholds associated with target PLs. In the not desirable case of having to use linear elastic analysis, heuristic definitions of target PLs on the simplified capacity curve are necessary.

The seismic input is defined by the hazard curve, obtained through a Probabilistic Seismic Hazard Analysis (PSHA), which gives the selected Intensity Measure (IM) as a function of the annual probability of occurrence (or the return period). Possible IMs are: peak ground acceleration (PGA), spectral acceleration for a given period, maximum spectral displacement, Arias intensity, Housner intensity (Douglas et al. 2014). In the standard case of nonlinear static analysis, the seismic demand is represented by an Acceleration-Displacement Response Spectrum (ADRS), which must be completely defined, for the specific site of the building under investigation, as a function of the assumed IM. In some cases a Vector-Valued PSHA can be used (Bazzurro and Cornell 2002), which gives a hazard surface instead of a hazard curve and allows a better description of the characteristics of the seismic input. When nonlinear dynamic analyses are performed, a significant number of proper records must be selected.

The outcome of the assessment is IM_{PL}, which is the maximum value of the intensity measure that is compatible with the fulfillment of each target PL (see Sect. 4); through the hazard curve, it is possible to evaluate the annual rate of exceedance λ_{PL} of the earthquake correspondent to this performance (or its return period $T_{R,PL} \approx 1/\lambda_{PL}$). These values are compared with the target earthquake hazard levels $\left(\bar{T}_{R,PL} \approx 1/\bar{\lambda}_{PL}\right)$, defined for the assessment as a function of asset characteristics, in terms of safety and conservation requirements.

The complete methodological path for the assessment of cultural heritage assets is based on three main steps (Fig. 2). The first one includes: 1) classification of the architectural asset and contained artistic assets; 2) definition of performance limit states and related safety and conservation requirements; 3) evaluation of seismic hazard and soil-foundation interaction; 4) knowledge of the structure (non-destructive testing, material parameters, structural identification) and definition of Confidence Factors (CF) by a sensitivity analysis. The second step is related to: 1) finalization of structural models for seismic analysis of the masonry building and the contained artistic assets (with identification of PLs); 2) verification procedures. Finally, in the third step, rehabilitation decisions are taken and, if necessary, the second step is repeated for the design of strengthening interventions.

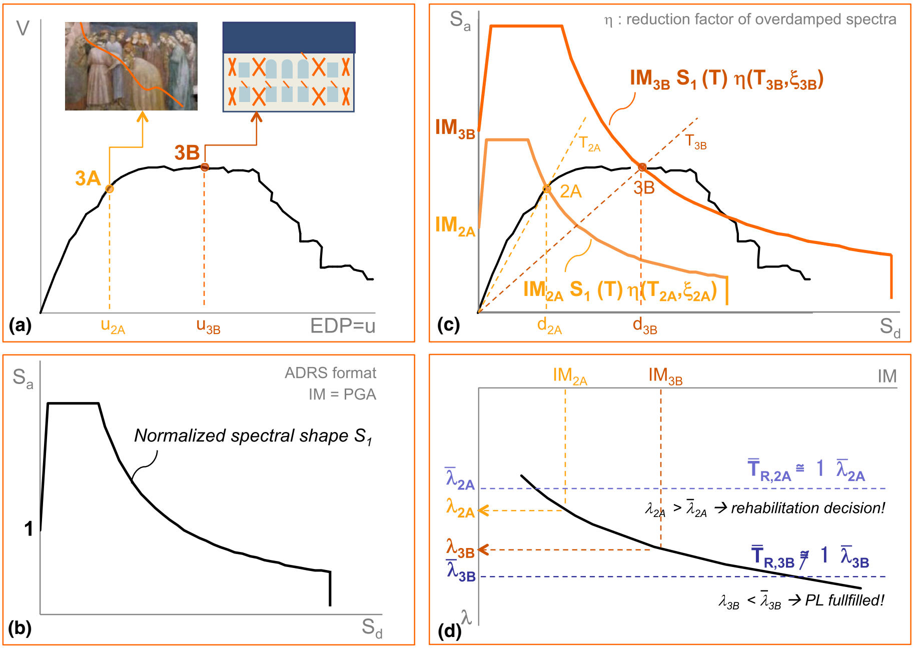


Fig. 1 The performance-based assessment of architectural and artistic assets: **a** execution of the nonlinear static analysis and positioning of PLs on the pushover curve (EDP = u; PLs = 3B and 2A); **b** definition of the seismic demand and selection of the representative IM (IM = PGA); **c** evaluation of IM_{PL} for the selected PLs; **d** verification (through the PSHA) and rehabilitation decisions

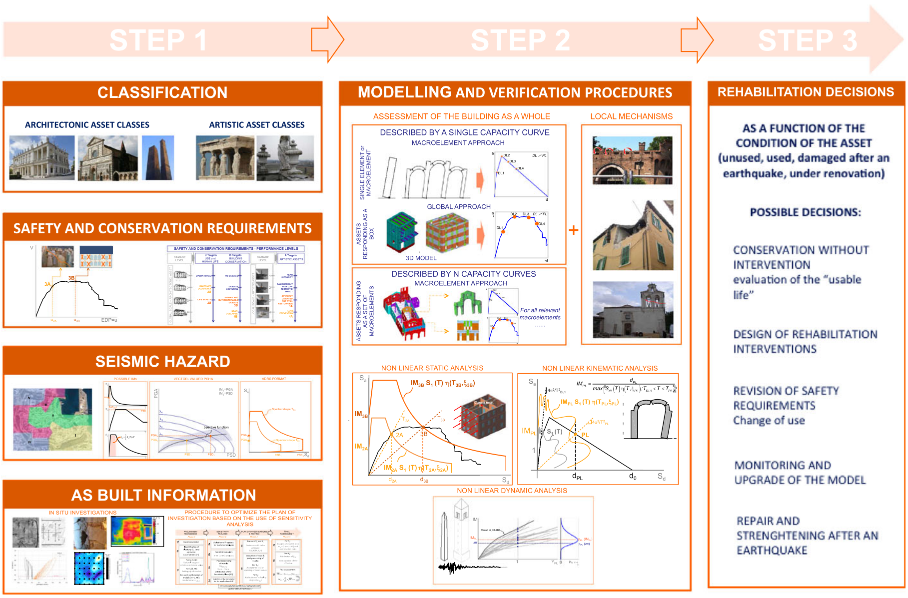


Fig. 2 Layout of the PERPETUATE procedure for the performance based assessment

	ARCHITECTURAL ASSET CLASS	MODEL TYPE			
		CCLM	SEM	DIM	MBM
A	Assets with a box behaviour Palaces, castles, religious houses, caravansaries, collective buildings		Global		Local
B	Assets analysable by independent macroelements Churches, mosques, modern theatres, markets, industrial buildings				
C	Assets characterized by monodimensional masonry elements Towers, bell towers, minarets, lighthouses, chimneys				
D	Arched structures subject to in-plane damage Triumphal arches, aqueducts, bridges, cloisters				
E	Massive structures with prevailing local failure of masonry Fortresses, defensive city walls, Roman and Greek theatres				
F	Blocky structures subjected to overturning Columns, obelisks, trilithes, archaeological ruins, Greek temples				

CCLM: Continuous Constitutive Law Models - SEM: Structural Elements Models – DIM: DIscrete Models – MBM: Macro Blocks Models

Standard Possible Rare

Fig. 3 Classification of architectural assets and related types of model for the seismic analysis

3 Classification of the architectural asset and contained artistic assets

The classification of architectural assets (Lagomarsino et al. 2011) is strictly related to the different types of seismic behavior, considering both building morphology (architectural shape, proportions) and technology (type of masonry, nature of horizontal diaphragms, effectiveness of wall-to-wall and floor-to-walls connections). To this end, the fundamental requirement is a proper knowledge of the building, which can be achieved by an interdisciplinary approach (historical information, survey and investigations).

Six architectural classes, from A to F, are defined (Fig. 3); it is worth noting that for each class, the list of building types in the second row is only explanatory, but the correct attribution of the class to a building is not related to its function.

The attribution of the architectural class to the examined building leads to the choice of the most suitable mechanical model to be adopted for the assessment; models (Fig. 4) are classified with reference to modeling scale (masonry material or structural elements) and type of discretization (continuous or discrete) (Calderini et al. 2010):

– CCLM (Continuous Constitutive Law Models): finite element modeling with phenomenological or micromechanical homogenized constitutive laws;
– SEM (Structural Elements Models): equivalent frame modeling by discretization in terms of piers, spandrels and other linear and nonlinear elements;
– DIM (Discrete Interface Models): discrete modeling of blocks and interfaces;
– MBM (Macro-Blocks Models): use of upper bound theorem of limit analysis to a predefined collapse mechanism of rigid blocks, under hypotheses compatible with the behavior of masonry structures (Heyman 1966).

Figure 3 shows the correlation between architectural classes and model types, in terms of frequency of use (if standard, possible or rare).

In this step of the assessment it is necessary to evaluate if the seismic behavior of the building can be represented by a single model or by a set of different models (Fig. 5). To this

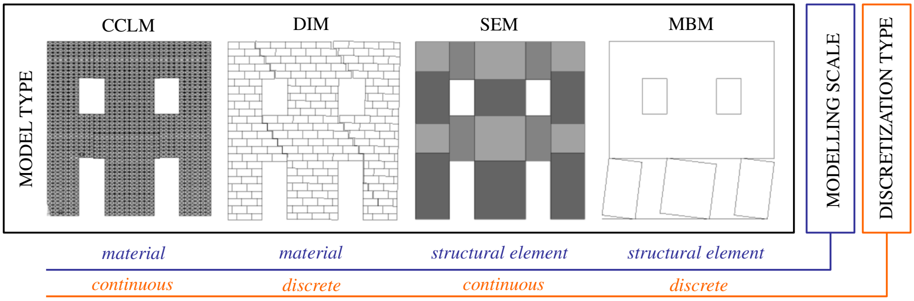


Fig. 4 Classification of modeling strategies

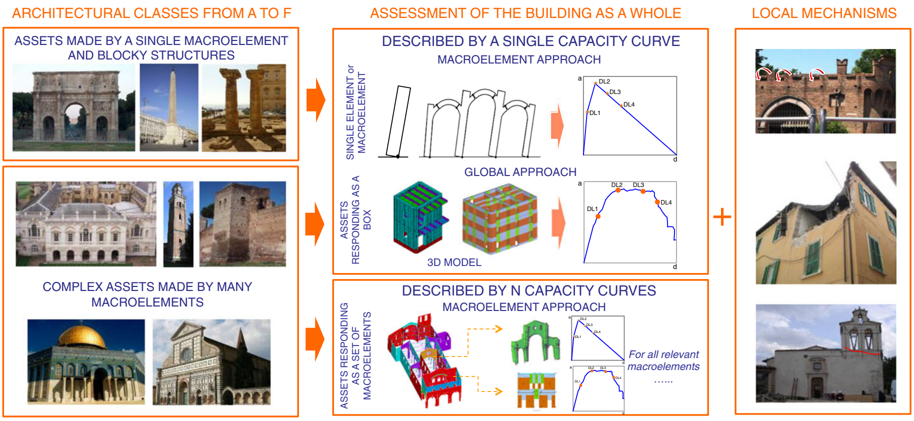


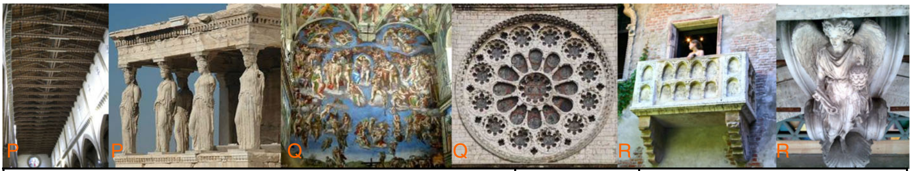
Fig. 5 Definition of models for the evaluation of the capacity at global and local scale

aim, it is useful to introduce the definition of macroelement: part of an architectural asset in which the seismic behavior can be analyzed independently from the rest of the structure. A macroelement may include a set of structural elements (i.e. piers and spandrels in the case of a wall) or coincide with the structure itself.

Thus, a single model is adopted in the case of assets made by a single element (such as those belonging to classes C, D and F) or by connected macroelements characterized by a global behavior (such as those of class A, which present the so called “box-type” behavior). On the contrary, the adoption of a set of models is suitable for complex assets, made by macroelements that behave quite independently; in this case the assessment requires to develop more than one model, even of different types (it is typical for assets of class B), and the result of the analyses in each macroelement must be then properly blended, in order to define the seismic assessment of the whole asset.

Moreover, it is necessary to identify the possibility of suffering local seismic mechanisms, which involve only a fraction of the total mass and cannot be properly considered by the global model. These mechanisms have to be studied using proper local models, usually involving out-of-plane behavior of small masonry portions. It is worth noting that an accurate identification of these mechanisms is possible only after a detailed survey of structural details of the building (step 4, Sect. 6).

The relevant immovable artistic assets have to be identified in the building, establishing also the interaction with the seismic response of the architectural asset; to this end a classi-



ARTISTIC ASSET CLASS		Interaction		MODEL TYPE			
		YES	NO	CCLM	SEM	DIM	MBM
P	Artistic structural elements Caryatid, carved stone columns, decorated wooden beams…	X					
Q	Artistic strictly connected to a structural element Carved stone plates, frescos, mosaics, stuccoes, decorated tiles,…	X					
R	Artistic assets that are independent elements Pinnacles, altars, sculptures, pulpits, Balconies, shelves, ….		X				

Standard Possible Rare

Fig. 6 Artistic assets classification and related modeling strategies

fication is proposed (classes P, Q and R presented in Fig. 6). The assessment in the case of artistic assets of Class P (artistic structural elements) or Q (artifacts strictly connected to a structural element) is directly related with the results of seismic analysis of the architectural asset, because a relation can be established between the structural damage and the consequence to the artistic element; in some cases, when the conservation of the artistic asset is very important, the static interaction between structural and non-structural elements can be investigated by a detailed finite element model. On the contrary, in the case of Class R (artistic assets that are independent elements), a specific independent model can be developed, which makes reference to the same approaches proposed for architectural assets; however, in this case, it is necessary to evaluate the response, in terms of ADRS, at the level where the artistic asset is placed, induced by the seismic action, which depends on the filtering effect of the main building (e.g. floor response spectrum, see Sect. 7.4).

It is worth noting that only artistic assets that are really relevant for the conservation, due to their value, should be considered in the seismic assessment. In fact, the need for preservation of an artistic asset can influence significantly the strengthening strategies because, on one hand, some techniques are not acceptable in the presence of artistic assets (e.g. mortar injection of frescoed walls with valuable paintings) while, on the other hand, strong reinforcement of single structural elements might be necessary for the preservation of the artistic asset, even if they are invasive interventions for the architectural asset.

Finally, the use of the building must be known for the seismic assessment. The function can be private, public or strategic; the activities can determine a rare, frequent or continuous occupation, in some cases with the possibility of crowding. The aim of the assessment can be different, depending on the current condition of the architectural asset:

– unused (sometimes even in ruin condition) and a decision upon the possibility of retrofitting has to be taken;
– damaged after an earthquake (or due to other actions) and repair/reconstruction interventions are necessary;
– under renovation (restoration of artistic assets; refurbishments) and it is necessary to consider also the need of structural protection from seismic risk;
– used and it is necessary to know if occupants can carry out activities in safe conditions.

4 Safety and conservation requirements

The implementation of PBA for cultural heritage assets needs to extend the rehabilitation requirements usually adopted for ordinary buildings, related to use and safety of people, also to conservation requirements, related to architectural and artistic cultural value. Therefore, target PLs are defined with reference to three different groups of *Safety and Conservation* requirements ($n = U, B, A$):

- *Use and human life* (*U*): also for a cultural heritage asset, similarly to ordinary buildings, the possibility of an immediate occupancy after an earthquake and the protection of human life have to be considered;
- *Building conservation* (*B*): due to the intangible value of a cultural heritage asset, the preservation from building damage is not related, as for ordinary buildings, to the costs of repair or rebuilding but to the possibility of restoration or to the collapse prevention, in order to maintain, at least, the monument as a ruin;
- *Artistic assets conservation* (*A*): in many cases, severe damage to artistic assets occurs also in the case of moderate damage to structural elements; moreover, damage to artistic assets is related to the local scale, while the other two groups of PLs (*U* and *B*) are related to the whole architectural asset; therefore, it is necessary to define specific PLs for each relevant artistic asset in the building.

PLs are obviously correlated to the seismic response of the structure, which is empirically defined, in macroseismic post-earthquake assessment (Grunthal 1998), by observational Damage States (DS): 1) slight; 2) moderate; 3) heavy; 4) very heavy; 5) collapse. The behavior of each architectural class is described by one or more EDPs, which are able, through proper thresholds, to identify Damage Levels (DL) on the pushover curve; DLk (k = 1,…,4) is the point after which the building experiences DSk. For complex assets (e.g. Class A) a multiscale approach is necessary to properly define DLs, by considering the behavior of single elements (local damage in piers and spandrels), macroelements (drift in masonry walls and horizontal diaphragms) and of the entire building (normalized total base shear, from global pushover curve). This topic is treated in step 5 (Sect. 7).

A first approximation is to establish a direct correlation between DLs and PLs: the names of target PLs are shown in Fig. 7, which are identified by an alphanumeric code (*kn*) that combines the corresponding DL ($k = 1, \ldots, 4$) and the type of safety and conservation requirement ($n = U$, B or A). For example, Life Safety (*3U*) is associated with heavy damage threshold (DL3), because it is assumed there are very few casualties or injured people with this damage level. The analogy with target PLs proposed in ASCE/SEI 41–13 (2014) and Eurocode 8—Part 3 (EN 1998–3 2005) is evident.

From a probabilistic point of view, the attainment of the threshold that corresponds to DLk means the probability of being in a DS greater of equal to k is 50 %; probabilities of occurrence of DSs are obtained by fragility curves. By using statistical correlations between DSs and losses (in terms of casualties and injured people, homeless, costs of repair), derived from post-earthquake assessment (e.g. in Coburn and Spence 2002), a refinement of acceptance criteria is possible. For example, Life Safety (*3U*) is not ensured in case of very heavy damage (DS4) or collapse (DS5), thus *3U* could be associated to the point of the capacity curve to which a given small probability of being in DS4 or DS5 is associated; according to this approach, d_{3U} threshold on the capacity curve might be anticipated with respect to DL3 (this occurs when DL3 and DL4 thresholds are very close). More information on this point is given in Sect. 9.

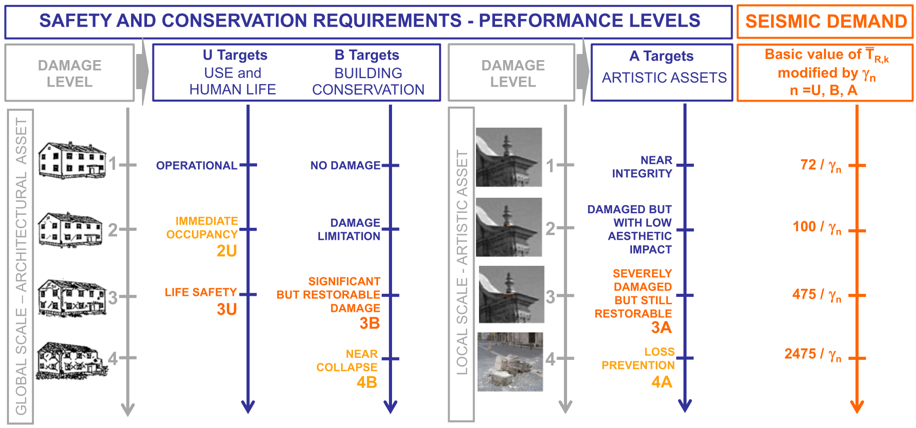


Fig. 7 PERPETUATE performance levels, corresponding damage levels and related target return periods: for each target, the primary and secondary PLs are marked in *orange* and *light orange*, respectively

For each DLk ($k = 1, \ldots, 4$) a related earthquake hazard level is assumed, expressed in terms of target return period $\bar{T}_{R,k}$ ($k = 1, \ldots, 4$) or annual rate of exceedance $\bar{\lambda}_{R,k} \approx 1/\bar{T}_{R,k}$. The same values adopted in the above-mentioned standards are proposed, except for DL2, for which a return period of 100 years is assumed, instead of 225 years; this corresponds to accepting a probability of occurrence of 40 % in 50 years instead of 20 %. This value is similar to the reference value for the design of new building according to Eurocode 8 (EN 1998–1 2004), which for Damage Limitation requirement considers $T_R = 95$ years (probability of 10 % in 10 years). The motivation of this departure from International standards on existing buildings is that ancient masonry buildings suffer moderate damage even for low intensity earthquakes (due to the negligible tensile strength of masonry), but the occurrence of some cracking is not detrimental for the preservation of the cultural heritage asset. Thus, the adoption for DL2 of a too demanding hazard level would require in most of the cases relevant retrofitting, which conflict with the principle of "minimum intervention".

The target return periods for each PL (k_n) is obtained from the return period $\bar{T}_{R,k}$ by applying importance coefficients (γ_n) that take into account the conditions of use (public, strategic) and the architectural and artistic value of the examined building:

$$\bar{T}_{R,kn} = \gamma_n \bar{T}_{R,k} \quad (k = 1, \ldots, 4 - n = U, B, A) \tag{1}$$

For each group of requirements, the primary and secondary PLs are identified (Fig. 7). It is recommended that the verification of the primary PLs be mandatory, while the secondary ones have to be considered only for relevant situations ($\gamma_n > 1$). In general, it is assumed that if the primary (and eventually the secondary) PLs are fulfilled, the remaining ones are fulfilled, too.

It is worth noting that artistic performance levels have to be considered only if relevant artistic assets are present in the building. In this case DLs refer only to the element (or macroelement) where the artistic assets are located. In general, the position of the corresponding displacement on the pushover curve (u_{kA}) may differ significantly from those related to Use and human life and Building conservation.

5 Seismic hazard

The seismic hazard assessment can refer to one or more intensity measures and the seismic input can be provided in different forms, depending on the classification of the building and the modeling strategy (Douglas et al. 2014).

Peak Ground Acceleration (PGA) is the most frequently adopted IM, due to the large amount of information (strong motion records) and models (Ground Motion Prediction Equations—GMPEs) that are available; it is a good parameter in the case of masonry buildings (Class A), due to their relatively short natural period, or massive structures (Class E). For assets of Classes A, B, C and D the spectral acceleration for a significant period of vibration of the asset ($S_a(T_{DLk}),\ k = 1$ or 2) may be a good IM. If the asset is characterized by a large displacement capacity, integral measures of the seismic input, like for example the Housner intensity I_H, provide good results, as they are representative of a wide range of the frequency content; it is the case of masonry buildings with prevailing rocking failure modes or of assets of Class F. For very slender assets of Class F (obelisks, single or multi-drum columns, etc.) a good intensity measure is the Peak Spectral Displacement (PSD).

Probabilistic Seismic Hazard Assessment (PSHA) needs information on the characteristics of the seismic sources that can affect the site (in terms of fault mechanism, depth, magnitude, recurrence times) and proper GMPEs that represent how *IM* attenuates from the epicenter to the site. The result is the hazard curve (Fig. 1), a relation between the maximum *IM* of the seismic input and the annual rate of exceedance λ (or the earthquake return period T_R). If more than one IM are considered, a Vector-Valued PSHA gives the hazard surface (Fig. 8); however, possible combinations of IMs are located in a restricted domain and, as the procedure proposed in PERPETUATE guidelines is not fully probabilistic and is based on the evaluation of the return period which is compatible with the fulfillment of the considered PL, a bijective function between the two IMs must be assumed.

Seismic input can be described by: 1) Acceleration-Displacement Response Spectrum (ADRS), completely defined for the specific site of the building under investigation as a function IM; 2) a proper set of time histories, selected from real recorded accelerograms or obtained through numerical modeling of the fault mechanism and the propagation towards the site. ADRS is necessary when nonlinear static (pushover) analysis is adopted, which is the standard method for a displacement-based assessment. Time histories are necessary for nonlinear dynamic analysis, which is a very effective method for assets of Class F; moreover,

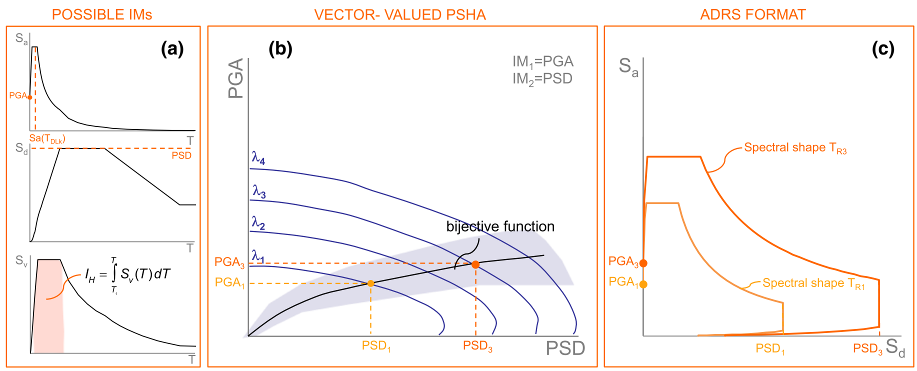


Fig. 8 Seismic input definition in terms of ADRS format: **a** possible IMs to be adopted as reference; **b** use of vector-valued PSHA; **c** ADRS format for nonlinear static procedures

it is useful when an accurate assessment is requested, as it may represent a validation and refinement of results given by static nonlinear analysis.

The Acceleration-Displacement Response Spectra (ADRS), wherein $S_d(T) = S_a(T)T^2/4\pi^2$ (being S_d and S_a the displacement and pseudo-acceleration response spectra, respectively), may be defined: 1) analytically, as in seismic codes; 2) through a piecewise linear function, by spectral acceleration values $S_a(T_h)$ for a given set of periods T_h $(h = 1, \ldots, N)$, obtained from GMPEs that already includes the soil amplification effects. The former type of spectra is defined by a set of parameters, which are dependent on the soil type and the magnitude; for example, starting from the Eurocode 8 (EN 1998–1 2004) format, new amplification factors and soil categories are proposed by Pitilakis et al. (2012, 2013) on the basis of SHARE's global strong-motion database (http://www.share-eu.org/). All parameters defining the ADRS are dependent on the return period and can be evaluated by considering the deaggregation of seismic sources in the PSHA. Despite this, usually PGA is assumed as IM, while other parameters are considered constant with the annual rate of exceedance λ.

For architectural assets that are very flexible (e.g. those of Class F), the shape of ADRS used by standards that adopt a force-based design, linked to hazard maps in PGA, is not accurate enough and has to be modified in the long periods range. Hazard maps in terms of PSD were developed for Italy by the INGV-S5 project (Faccioli and Villani 2009) and the shape of displacement response spectra has been discussed by Faccioli et al. (2004). If both PGA and PSD hazard curves are available (which means the bijective relation between the two IMs is known), the shape of the acceleration response spectrum changes with the return period and can be defined by properly modified relations (e.g. Lagomarsino 2014).

If nonlinear dynamic analyses have to be performed, the seismic input is defined through a proper set of acceleration time histories. Usually the best option is to select them from recorded digital accelerograms; the selection is possible from strong motion databases (Smerzini et al. 2013; Iervolino et al. 2009) and must refer to parameters like: magnitude, fault mechanism, epicentral distance, soil condition at the site. The first two parameters may be obtained from PSHA, by a deaggregation of the contributions of the different seismic sources, aimed to single out the characteristics of the earthquake that contribute most to the seismic hazard for the target return period related to the PL under investigation.

The minimum number of records is related to the adopted verification procedure. Incremental Dynamic Analysis (IDA) is based on scaling strong motion records, with the aim of evaluating the IM that is compatible, for each time history, with the PL under investigation (Vamvatsikos and Cornell 2002). Cloud method consists in performing nonlinear dynamic analyses with many records, without any scaling; thus, in order to evaluate the mean value of IM which is compatible with the fulfillment of a specific PL (that is IM_{PL}), it is necessary to have a sufficient number of cases to apply the Multiple Stripe Analysis (MSA), in particular which produces a seismic demand very close to this condition (Jalayer and Cornell 2009).

6 As-built information

The aim of as-built information process is the acquisition of knowledge for defining the structural model of the building and artistic assets (if present), with reference to: geometry of structural elements; foundations; material properties; historical data on transformation and damage; state of maintenance and damage mechanisms identification (in case of post-earthquake assessment); dynamic behavior. However, in case of historical masonry buildings

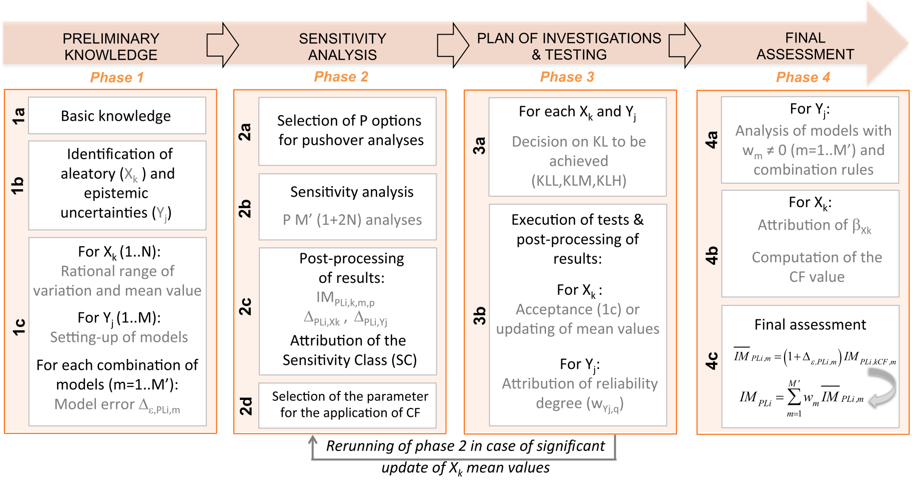


Fig. 9 Flowchart of the proposed procedure for the use of sensitivity analyses to define the investigation plan and CF (Cattari et al. 2014b)

it is necessary to consider that the number of investigations should be minimized for reducing the impact on conservation, as well as the costs.

In order to consider in the assessment the uncertainties due to incomplete knowledge, the common approach adopted by standards for existing structures (EN 1998–3 2005; ASCE/SEI 41–13 2014) or, more specifically, for cultural heritage assets (Recommendations P.C.M. 2011) is based on the definition of a discrete number of Knowledge Levels (KL), achievable as a function of gathered information, and on the application of a Confidence Factor (CF) to one parameter of the analysis, assumed *a priori* as the most affecting the assessment.

Within this context, the innovative contribution of the PERPETUATE guidelines is the introduction of sensitivity analysis as essential tool for the seismic assessment of existing and monumental buildings (Cattari et al. 2014b). In particular, it allows improving some fundamental issues as:

– to identify the parameters that most affect the structural response, allowing to optimize the investigation plan and strengthen the link between knowledge and assessment;
– to explicitly include in the methodological path the evaluation of uncertainties, by considering both aleatory (treated as random variables) and epistemic (treated by the logic tree approach) ones, as well as the model error contribution;
– to properly select (instead of *a priori*) the parameter for the application of CF and calibrate its value (instead of assuming it conventionally).

The use of sensitivity analysis is codified in a well-defined procedure, subdivided into four steps (Fig. 9): 1) preliminary knowledge; 2) sensitivity analysis; 3) plan of investigations and execution of tests; 4) final assessment.

One of the main problems in numerical modeling and verification of ancient buildings is the availability of reliable mechanical parameters of masonry, both because of the invasiveness of in-situ testing and the not negligible intrinsic error of measurements. Reference values of the main mechanical parameters of masonry (elastic modulus, shear and compressive strength, panel drift limits) are provided for a wide list of stone and brick masonry types, based on available data from literature and new experimental tests (Krzan et al. 2014).

In order to improve the knowledge of the architectural asset, with the aim of a more reliable modeling and assessment, ambient vibration tests can be very useful for identifying the overall dynamic behavior of heritage buildings and the connections between macroelements (Karatzetzou et al. 2014).

Finally, in case of complex assets, the simulation in laboratory by a scaled or full-scale mockup can be very useful. The mockup can reproduce the whole asset or a single macroelement. In particular shaking table testing turns out to be most effective solution (De Canio et al. 2014a, b; Drosos and Anastasopoulos 2014).

7 Structural models for the seismic analysis and assessment procedures

The outcome of the PBA proposed in PERPETUATE project is the maximum seismic intensity measure IM_{PL} compatible with the fulfillment of each performance level, which is identified in the second step (safety and conservation requirements—Sect. 4). To this aim, the following methods of analysis and verification procedures are considered:

- Nonlinear Static Analysis (pushover) and Capacity Spectrum Method (CSM), based on the comparison between the displacement demand, obtained by a properly reduced acceleration-displacement response spectrum, and the displacement capacity.
- Nonlinear Dynamic Analyses, based on a statistical evaluation of IM_{PL} by using a large amount of records (cloud method), analyzed by the Multiple Stripe Analysis (MSA), or a proper selection of time-histories, scaled in order to perform an Incremental Dynamic Analysis (IDA).

The first method (CSM) is assumed as the standard one. It can be used for all classes of architectural assets and also for the assessment of artistic assets. The pushover curve is obtained according to well known procedures (definition of load pattern, mixed force-displacement incremental analysis), widely applied for modern structures; the application to irregular structures with flexible horizontal diaphragms poses some questions and some specific hints for the different classes of cultural heritage assets are examined in depth in Calderini et al. (2012).

The second one (by IDA or MSA), even if more accurate, is suggested only for some classes of assets (e.g. Class F), for which it is applicable with a reasonable computational effort; it can be used also as validation of CSM results, in order to improve the reliability of the assessment.

It is worth noting that linear elastic analysis may be considered as possible alternative only in case of very complex assets for which nonlinear analyses are not feasible (e.g. complex assets in Class B). In these cases, instead of referring to the use of a behavior factor, it is possible to define a simplified capacity curve (Cattari and Lagomarsino 2012). The spectral acceleration capacity, obtained by the linear elastic analysis, can be considered as representative of DL1, and the corresponding initial period T_{DL1} is obtained. Then the capacity curve is obtained by assuming: 1) the equivalent lightly cracked period T_{DL2}; 2) an overstrength ratio (in order to define the spectral acceleration capacity at DL2); 3) displacement capacities for DL3 and DL4. The assessment is then made by the CSM.

According to what was stated in Sect. 3, the seismic assessment has to consider both the building response as a whole (global response—$IM_{PL,G}$) and the possible occurrence of local mechanisms ($IM_{PL,L}$).

In the following, after some general issues related to structural modeling (Sect. 7.1), the detailed procedure for the seismic assessment through nonlinear static approach is described in Sect. 7.2; some hints on nonlinear dynamic assessment are provided in Sect. 7.3, while in Sect. 7.4 the verification of local mechanisms is treated. Finally, in Sect. 7.5 some specific issues related to the seismic assessment of artistic assets are illustrated.

7.1 Issues related to structural models

Structural modeling is part of the knowledge phase (Sect. 6) as, through the sensitivity analysis, it helps in the definition of the investigation protocol. Four classes of mechanical models are available (Fig. 4), characterized by different levels of complexity, which can be adopted for the various classes of architectural assets (see Sect. 3, Fig. 3).

Structures that belong to classes C, D and F are usually made by a single macroelement and are described by a single model.

Complex architectural assets, such as those belonging to classes A and B, can lead to two alternative modeling approaches (Fig. 5):

– buildings characterized by box behavior: in this case a 3D model of the whole building is appropriate (*global scale* approach);
– buildings made by a set of N_m macroelements, which exhibit an almost independent behavior: each macroelement is modeled independently (*macroelement scale* approach) and the seismic load needs to be assigned by a proper redistribution; in this case the assessment of the asset as a whole is then made through proper combination of results achieved in each macroelement (see Sect. 7.2.5).

The *global scale* approach is typical of buildings of class A but can be sometimes adopted also for architectural assets of class B, when macroelements are well connected. The *macroelement scale* approach is necessary for most of structures of class B, but also for very few buildings of class A, when horizontal diaphragms are flexible and/or internal walls are sparse.

Once the model has been selected, a set of possible options can be considered; the results of sensitivity analysis and in-situ investigations allow to select the necessary degree of accuracy for a reliable assessment. CCLM and DIM models are able to describe the structural behavior in detail, while SEM and MBM models require some preliminary assumptions, which are not always straightforward. For example, in the case of SEM the dimensions of piers and spandrels, as well as the topology of the equivalent frame, are not unique, while in the case of MBM, often there are different possible subdivisions of the macroelement into a kinematic mechanism of rigid blocks.

Particular attention should be paid to modeling strategies for masonry foundations and soil-structure interaction, because historical masonry buildings are massive structures and often the foundations are not so deep. For slender building typologies, such as towers, the Soil-Foundation-Structure Interaction (SFSI) may produce significant rocking effects and associated damping in the system. For massive high frequency structures the importance of such interactions effects may be equally important, because, due to their stiffness, the support to the ground can not be considered rigid. Proper impedance functions have been developed by PERPETUATE project on the basis of detailed numerical analyses (Pitilakis and Karatzetzou 2014), which can be implemented in the model (in particular SEM).

7.2 Nonlinear static analysis and capacity spectrum method

The procedure aimed to evaluate $IM_{PL,G}$ may be summarized in the following main steps: 1) execution of the pushover analysis; 2) identification of the DLs, and related PLs, on the pushover curve; 3) conversion of the pushover curve into capacity curve; 4) given the seismic demand (in terms of proper IM), computation of the *IM* value compatible with the examined PL ($IM_{PL,G}$). The four steps are described in the following sections in the case of a global scale approach, while in Sect. 7.2.5 the case of a macroelement scale approach is treated (assessment based on a set of N capacity curves).

7.2.1 Pushover analysis

The pushover analysis can be carried out, in most cases, using the modeling approach SEM, because of the relatively limited number of degrees of freedom even for complex assets. CCLM and DIM can be adopted only for simple structures (Class C and D) or single parts (macroelements) of complex structures (Class A and B). For assets of Class F the MBM can be used; in this case, the pushover curve is obtained through a kinematic limit analysis, taking into account geometric nonlinearities, by applying to the seismic masses a pattern of horizontal loads and incrementing the mechanism displacements of each block.

The execution of pushover analysis requires proper choices concerning: i) seismic load pattern; ii) selection of control node (to optimize the numerical convergence); iii) representative displacement to be considered in the pushover curve.

Regarding load pattern (i), possible options are (Aydinoglu and Onem 2010): 1) proportional to masses (*uniform*); 2) proportional to the fundamental modal shape (*modal*); 3) given by a proper combination of different modes (SRSS-based or CQC-based); 4) proportional to the product mass × height (*pseudo-triangular*); 5) adaptive load pattern (*adaptive*).

Usually codes propose to assume at least two patterns, because the inertial force distribution changes, with the occurrence of damage, from an initial *modal* distribution to patterns that are proportional to the deformed shape, which at collapse is closer to the *uniform* one. A promising alternative is the *adaptive pushover*, in which at each step of the analysis the load pattern is updated as a function of the evolution of the nonlinear response of the structure (Antoniou and Pinho 2004; Chopra et al. 2004; Gupta and Kunnath 2000). However, very few applications to masonry structures can be found in the literature (Galasco et al. 2006), due to their distinctive features, such as the softening response of masonry under shear and the presence of flexible floors.

The *modal* pattern is not reliable in the case of flexible horizontal diaphragms, because each mode mainly involves the local behavior of single walls, having a very low fraction of the participating mass. Thus, in order to reach a significant total mass participation, a SRSS-based or CQC-based load pattern can be defined, by considering the first N_r modes in a given direction, being characterized by a constant sign of displacements at different levels in each wall; if the resulting participating mass is still lower than 75 %, this percentage should be anyhow considered in the conversion to the equivalent SDOF system. If the building is regular in elevation, a simpler alternative is the use of a *pseudo-triangular* load pattern, because it assures that all the structural masses are involved in the pushover analysis.

An advanced approach, in order to treat the most complex configurations (flexible floors, irregularity in plan and in elevation), is the *multi-modal pushover* analysis (Chopra and Goel 2002), which can also combine, if necessary, the effect of both components of the input motion (Reyes and Chopra 2011), instead of considering them as independent.

The choice of control node (ii), both in elevation and plan, is important in order to optimize the convergence of the nonlinear pushover analysis. Regarding the elevation, it is suggested to select the control node above the level in which the collapse occurs. For this reason, codes commonly propose to assume the control node at the top floor. Regarding the in plan location, the choice represents a very crucial issue in case of existing buildings with timber floors or vaults. In fact, while in the case of rigid floors the results are almost insensitive to the position of the control node, in the case of flexible ones they strongly depend on it, because of the different stiffness and strength of masonry walls. The numerical results are more accurate if the control node is selected in the wall that collapses as the first.

The selection of the representative displacement for the pushover curve (iii) is a crucial point for the conversion into the capacity curve (Sect. 7.2.3) when diaphragms are not rigid and/or the building is irregular in plan. In fact, the capacity curve shows very different displacement capacity (ductility) whether the considered displacement is that of a wall that reaches failure or not. Thus, instead of the displacement of the control node, it is preferable to use the average displacement of all nodes at the same level, weighted by the seismic nodal mass. This procedure represents a heuristic approach useful to get an unambiguous outcome, which has also a physical interpretation: indeed, the displacement-based approach considers the capacity of seismic masses to move, in comparison with the earthquake displacement demand.

In conclusion, despite the above-mentioned difficulties and bearing in mind the lack of reliability of linear analysis for simulating near collapse conditions, nonlinear static analysis still remains the best possible option for the seismic assessment of complex masonry buildings.

7.2.2 Identification on pushover curve of damage levels and related performance levels

In order to check the fulfillment of the considered PL, the corresponding DL has to be positioned on the pushover curve, by using all information provided by the incremental nonlinear static analysis. This is a complex task, which is tackled by codes and recommendation documents according to the following main approaches:

- *Structural element approach*. It assumes that the attainment of a certain DL in the building corresponds to the step in which the first structural element reaches the same DL. This approach is adopted when the mechanical model is not able to capture the progressive strength degradation of the pushover curve.
- *Heuristic approach*. DLs are directly defined on the pushover curve on the basis of conventional limits, usually expressed in terms of interstorey drift and decay fraction of the overall base shear. This approach requires the use of shear-drift constitutive relations with strength degradation and limited ductility; it is adopted in the case of masonry buildings, where usually some elements reach a given DL much earlier than others.

In the case of complex cultural heritage buildings it is necessary to refer to mechanical models which are able to describe the stiffness and strength degradation, due to material and geometrical nonlinearities; however, the application of the *heuristic approach* may result quite conventional and not reliable if adopted as single criterion to define the DLs on the pushover curve, because it does not detect the occurrence of heavy DLs at local or macroelement scale (i.e. each single masonry wall). If the building is very large and horizontal diaphragms are flexible, a significant damage in one single wall may not appear evident in the pushover curve of the whole structure. The same applies for damage in structural elements, which can spread too much in the building without any tangible effect in the global pushover curve.

The definition of DLs in the PERPETUATE project is based on a multiscale approach that takes into account the asset response at different scales: structural elements scale (local damage), architectural elements scale (damage in macroelements) and global scale (pushover curve). Proper criteria have to be defined for each architectural class.

Let us consider, as an example, the assets of class A (*Assets with a box behavior*), which are usually complex buildings made by many macroelements, masonry walls and horizontal diaphragms. According to SEM models, vertical piers and horizontal spandrels may be identified in each masonry wall. Proper constitutive laws must be defined for these structural elements, even considering the strength degradation, and the attainment of progressing DLs is checked through given drift limits δ_{DLi} (being the damage levels DLi at element scale defined for i from 1 to 5).

Since the final seismic assessment is made through the global pushover curve, the displacement corresponding to attaining DLk ($k = 1, ..., 4$) is computed as:

$$u_{DLk} = \min\left(u_{E,DLk};\ u_{M,DLk}; u_{G,DLk}\right) \quad k = 1, \ldots, 4 \tag{2}$$

where $u_{E,DLk}$, $u_{M,DLk}$, and $u_{G,DLk}$ are the displacements on the pushover curve corresponding to the reaching predefined limit conditions at element (E, piers or spandrels), macroelement (M, e.g. each masonry wall and, eventually, horizontal diaphragms) and global (G) scales, respectively.

At global scale, the variable chosen to monitor the attainment of $u_{G,DLk}$ is the fraction κ_G of the total base shear over the maximum base shear of the pushover curve ($\kappa_G = V/V_{max}$); proper thresholds (κ_{DLk}) are defined for DL1 and DL2 on the ascending part of the curve while DL3 and DL4 are located on the descending one.

At macroelement scale, the following variables are adopted: in the case of masonry walls, the interstorey drift $\theta_{w,l}$ by any wall and level ($w = 1, \ldots, N_w$—wall number; $l = 1, \ldots, N_l$—level number) must not reach the threshold θ_{DLk}; in case of diaphragms, the angular strain $\gamma_{q,l}$ ($q = 1, \ldots, N_q$—diaphragm number) must not reach the threshold γ_{DLk}.

It is worth noting that usually the interstorey drift is computed only referring to the horizontal displacement, but this is correct only in the case of strong spandrels (shear-type behaviour). More in general, the interstorey drift of wall $\theta_{w,l}$ has to be evaluated taking into account for the contribution of both the horizontal displacement and rotation of nodes, for example according to:

$$\theta_{w,l} = \frac{\bar{u}_{w,l} - \bar{u}_{w,l-1}}{h_l} + \frac{\bar{\varphi}_{w,l} + \bar{\varphi}_{w,l-1}}{2} \tag{3}$$

where: h_l is the interstorey height at level l, while $\bar{u}_{w,l}$ ($\bar{u}_{w,l-1}$) and $\bar{\varphi}_{w,l}$ ($\bar{\varphi}_{w,l-1}$) are the average horizontal displacement and rotation of nodes located at level l (or $l - 1$) in wall w (positive if counterclockwise).

Finally at local scale, the cumulative rate of elements that reach a certain DLi (piers—$\Lambda_{P,DLk}$—and spandrels—$\Lambda_{S,DLk}$) is introduced to check for the attainment of $u_{E,DLk}$.

The cumulative rate of damage $\Lambda_{S,DLk}$ is defined as the percentage of spandrels that reached or exceeded a given DLi (checked through the reaching of given threshold of drift δ_{DLi} at structural element scale):

$$\Lambda_{S,DLk} = \frac{1}{N_S}\sum_{S} \mathrm{H}\left(\frac{\delta_s}{\delta_{DLi}} - 1\right) \quad i = k + 2 \tag{4}$$

where the sum is extended to the total number of spandrels ($s = 1, \ldots, N_S$) in the building and H is the Heaviside function (equal to 0 until the demand δ_s in the s-th spandrel does not reach the capacity δ_{DLi}).

Table 1 Criteria of the multiscale approach proposed for DLk in the case of global response of Class A

Scale	Variable	Thresholds	DL1	DL2	DL3	DL4
Local	$\Lambda_{P,DLk} - \Lambda_{S,DLk}$	Λ_{P-S}	0.025–0.05			
Macroelement	$\theta_{w,l}$	θ_{DLk}	0.0005–0.001	0.0015–0.003	0.0035–0.005	0.0055–0.007
Global	θ_G	θ_{DLk}	≥0.5	0.95–1	0.8–0.9	0.6–0.7

The cumulative rate of damage $\Lambda_{P,DLk}$ is defined as the percentage of piers that reached or exceeded a given DLi, weighted on the corresponding cross section A_p:

$$\Lambda_{P,DLk} = \frac{\sum_P A_p \mathrm{H}\left(\frac{\delta_p}{\delta_{DLi}} - 1\right)}{\sum_P A_p} \quad i = k + 1 \tag{5}$$

where the sum is extended to the total number of piers ($p = 1, \ldots, N_P$).

It is worth noting that, according to Eqs. (4) and (5), a higher damage level is accepted in spandrels than in piers. For example, to check the attainment of DL2 ($k = 2$) reaching of damage levels 3 ($i = k + 1$) and 4 ($i = k + 2$) is checked at the scale of pier and spandrel elements, respectively. In case of DL4, only attainment of damage level 5 in piers is considered. This assumption is due to the different hierarchic role of these elements in the behavior of masonry walls. In fact, piers represent the most important elements, which bear both static loads and seismic action, whereas spandrels are secondary elements, which connect piers by transmitting bending moments.

Table 1 proposes possible values of thresholds to be used for checks at the different scales; each threshold can be selected within a given range, which comes out from literature and expert judgment and could be validated or updated by further experimental tests or evidence from observed damage. It is worth noting that values of the variables that are not included in any range correspond to conditions in which the attainment of a given DS (Damage State) is sure. At local scale, the same value Λ_{P-S} can be used as threshold for cumulative variables $\Lambda_{P,DLk}$ and $\Lambda_{S,DLk}$ for both piers and spandrels and all DLk; it allows for damage spreading to a limited percentage of elements, but avoids attainment of DLk due to just a single element. At macroelement scale, interstorey drift limits may be selected within given ranges, which are compatible with values proposed in Calvi (1999). At global scale, range of values for the thresholds of the fraction of the maximum overall base shear are compatible with provisions in Eurocode 8, Part 3 (EN 2005); in the case of DL1, a lower bound is defined in order to avoid the occurrence of a slight DS in the very beginning of the ascending branch of the capacity curve. In some cases, additional checks at macroelement scale (e.g. for horizontal diaphragms) or local scale (e.g. by monitoring the damage is some relevant elements) may be considered for specific performance requirements (i.e. related to the safety of people).

Figure 10 illustrates synthetically the steps to be followed for the definition of DLk on the pushover curve, by the multiscale approach.

Proper heuristic criteria to define DLs on the pushover curve obtained by MBM models are also proposed, which have been validated through many nonlinear dynamical analyses performed on Housner model or bilinear degrading elastic systems (Lagomarsino 2014). The capacity curve is obtained, as illustrated in Sect. 7.2.3, from the conversion of the pushover curve evaluated under the hypothesis of rigid blocks; an ascending branch is added in order to describe the elastic behavior before the activation of the mechanism (the secant period T_{DL2} and, eventually, the elastic period T_{DL1} have to be estimated). Figure 11 shows how DLs

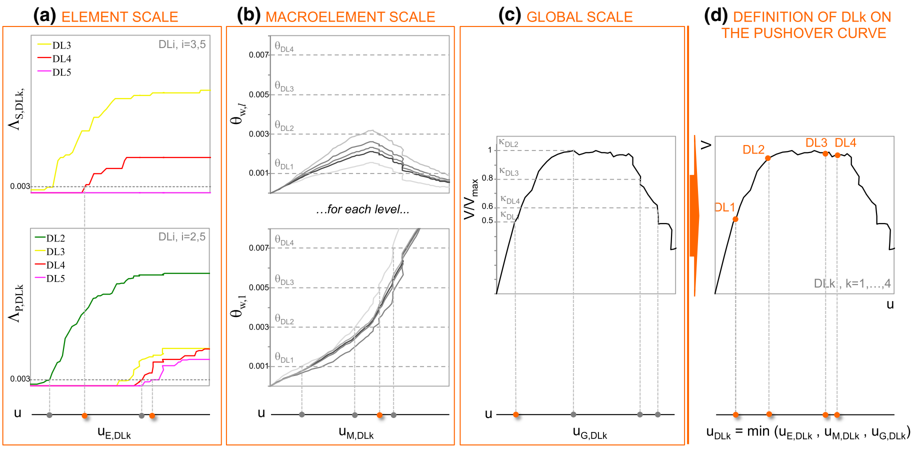
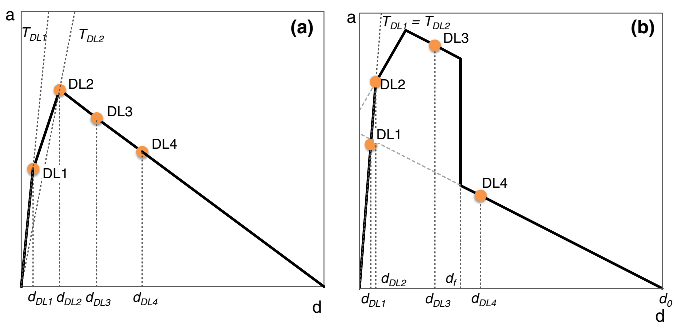


Fig. 10 Multiscale approach for the DLs identification in the case of buildings of class A: variables monitored at element (**a**), macroelement (**b**) and global (**c**) scales, respectively; **d** final position of DLk on the pushover curve as resulting from the worst condition at different scales examined



$d_{DL1} = 0.7 d_{DL2}$ if $T_{DL1} = T_{DL2}$
$d_{DL2} \le d_{DL3} = 0.25 d_0 \le d_f$
$d_{DL4} = 0.4 d_0$

Fig. 11 Criteria proposed for the definition of DLs in case of pushover curves obtained from MBM models (from Lagomarsino 2014): response of a single rigid block without (**a**) and with (**b**) tie-rod

can be identified on the pushover curve (Lagomarsino 2014) for two different typical cases. In addition, some criteria at element scale can be added, referring to local critical conditions that can occur with the progression of the mechanism (e.g. unthreading of timber joists or roof trusses).

7.2.3 Conversion of the pushover curve into the capacity curve

The pushover curve is converted into the capacity curve of the equivalent nonlinear single degree of freedom system (SDOF), which can be compared with the seismic demand (Fajfar 2000). It is based on the evaluation of a participation coefficient Γ and a participation mass m^*, which are obtained by assuming a reference deformed shape (EN 1998–1 2004, see Fig. 13).

In the case of nonlinear static analyses on multi-degree of freedom systems (e.g. by SEM or CCLM), the fundamental mode shape can be assumed as reference deformed shape, if the loading pattern is similar to the *modal* one. Otherwise, the deformed shape obtained by the pushover analysis at the first steps (i.e. in the initial elastic phase) can be used.

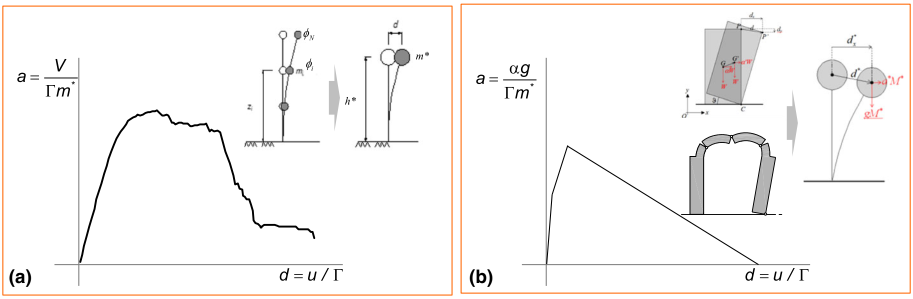


Fig. 12 Conversion of the pushover curve into capacity curve: **a** nonlinear static analysis (from CCLM/SEM/DIM models); **b** nonlinear kinematic analysis (from MBM/DIM models)

In the case of nonlinear kinematic analysis, by MBM, being already the mechanism a single degree of freedom system, the blocks displacement shape is assumed as deformed shape for converting the original pushover curve into capacity curve (Lagomarsino 2014).

Figure 12 summarizes the conversion of the pushover curve (expressed in terms of base shear V or collapse multiplier α and representative displacement u of the original MDOF system—see Sect. 7.2.1) into the equivalent capacity curve (expressed in terms of acceleration a and displacement d of the equivalent SDOF system) for the two above-mentioned cases.

7.2.4 Evaluation of IM values compatible with the attainment of each PL

Different methods are available in the general framework of the displacement-based assessment for the evaluation of the displacement demand on the capacity curve, given an Acceleration-Displacement Response Spectrum: i) the Capacity Spectrum Method (CSM) and the Displacement-Based Method (Freeman 1998; Calvi 1999; Priestley et al. 2007); ii) the N2 Method (Fajfar 2000); iii) the Coefficient Method (ASCE/SEI 41–13 2014); iv) Modified ADRS (MADRS) Method (FEMA 440 2005).

The method proposed in the PERPETUATE guidelines is the classical CSM, which uses overdamped spectra (based on the definition of a linear-equivalent system, considering the secant stiffness at the intersection between capacity and demand, with a proper equivalent viscous damping, consistent with the hysteretic dissipation due to nonlinear behavior). If the seismic input is given, the evaluation of the displacement demand requires an iterative procedure. On the contrary, the evaluation of the seismic input that produces a given displacement is straightforward, once the corresponding equivalent damping is known.

The procedure is thus simple and direct because for the evaluation of IM_{PL}, it is necessary to know the period (T_{PL}) and damping coefficient (ξ_{PL}) associated to each PL, as defined on the capacity curve.

The period T_{PL} may be easily computed as:

$$T_{PL} = 2\pi\sqrt{\frac{a_{PL}}{d_{PL}}} \tag{6}$$

The values of damping coefficient ξ_{PL} can be estimated by different approaches:

– from cyclic pushover analyses (Fig. 13), by evaluating the hysteretic dissipation ξ_h as a function of d, and adding an elastic viscous damping ξ_0 (assuming a value between 3 and 5 %, except for assets of Class F, which are characterized by lower values—e.g. 2%); ambient or forced vibration tests on the examined building can provide an estimate of ξ_0;

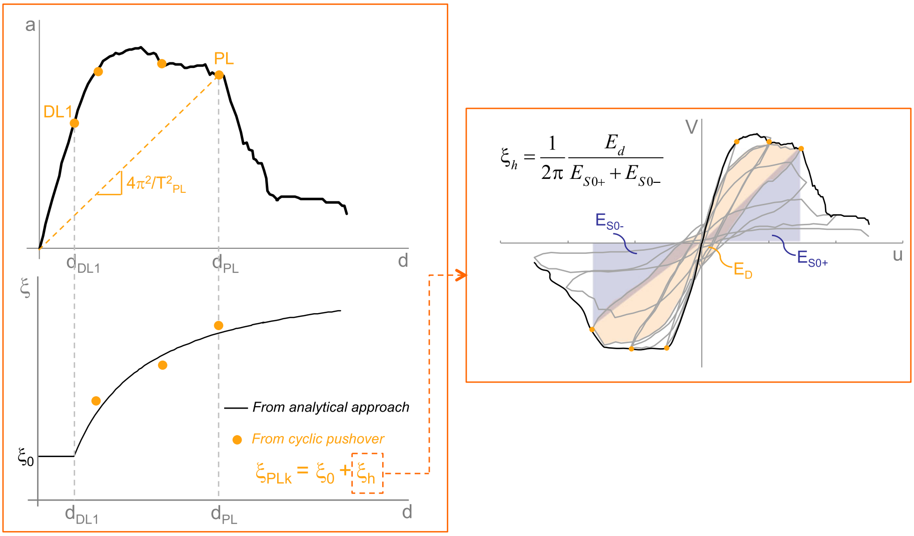


Fig. 13 Evaluation of equivalent viscous damping ξ_{PL} through analytical approach and use of cyclic pushover

Table 2 Reference values for $\xi_{h,max}$ and β for different classes of architectural assets

	Architectural asset class				
	A	B	C	D	F
$\xi_{h,max}$(%)	25	20	15	15	5
ζ	1.5	2	1.5	1.2	1

– from analytical expressions proposed in literature for similar assets (Calvi 1999; Priestley et al. 2007; Blandon and Priestley 2005; Sullivan et al. 2013); the following expression is suggested:

$$\xi_{PL} = \xi_0 + \xi_{h,\max}\left(1 - \frac{1}{\mu_{PL}^{\varsigma}}\right) \tag{7}$$

where: $\mu_{PL} = d_{PL}/d_{DL1}$ is the ductility; $\xi_{h,max}$ is the asymptote of the hysteretic damping and ς is a free parameter coefficient that influences the rate of increase of hysteretic damping with ductility (reference values are suggested in Table 2).

Finally, if only one *IM* is considered, the ADRS has a fixed shape and can be defined by introducing a displacement response spectrum normalized to *IM*:

$$S_d\,(T, IM, \xi) = IM\ S_1(T)\eta\,(T, \xi) \tag{8}$$

If the ADRS is regular and the spectral displacement increases monotonically with the period T (or remains constant), $IM_{PL,G}$ can be simply evaluated as the IM for which the spectral displacement demand $S_d(T_{PL}, IM, \xi_{PL})$ is equal to d_{PL} (Fig. 14a). In order to extend the CSM to the application in the case of irregular ADRS (Fig. 14b), generally defined as a piece-wise linear function, the following expression in proposed for the evaluation of

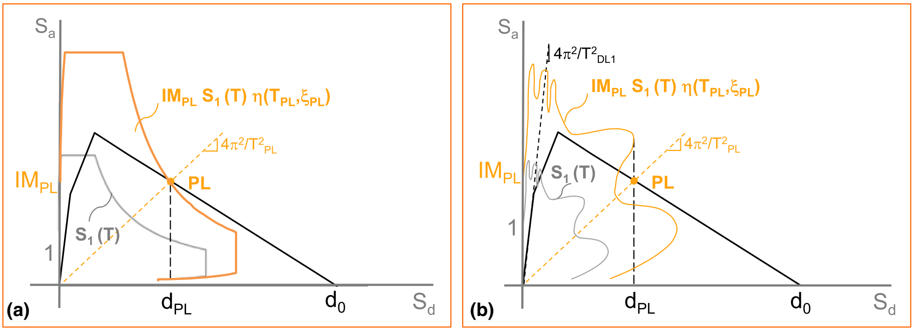


Fig. 14 General CSM procedure proposed for the evaluation of IM_{PL} in case of: **a** analytical ADRS; **b** piece-wise linear ADRS

$IM_{PL,G}$:

$$IM_{PL,G} = \frac{d_{PL}}{max\left[S_1(T)\,\eta(T, \xi_{PL}); T_{DL1} < T < T_{PL}\right]} \tag{9}$$

It is worth noting that this method for the PBA (nonlinear static analysis and CSM) should not be used with ADRS directly obtained from recorded time histories, because often their shape is very irregular. If the assessment is made by IDA with real accelerograms and a comparison with the results of static nonlinear analysis is needed, the mean ADRS from those of records can be used, which is always more regular and smoothed. Despite this, ADRS similar to the one of Fig. 14b may result from the seismic hazard analysis if the site is subjected to soil amplification phenomena, which are numerically modeled, or in the case of floor spectra, due to the amplification of seismic input at the upper levels of the building, used for the verification of local mechanisms or artistic assets.

7.2.5 Seismic assessment of complex assets (described by many capacity curves)

This case regards assets made by a set of macroelements that exhibit an almost independent behavior; it is typical of Class B and, rarely, of Class A, if horizontal diaphragms are very flexible and there are poor connections between walls (in this case the building does not have a box-behavior and it is not possible to describe it by a global capacity curve).

The seismic behavior of each of the N_m macroelements is analyzed independently and the assessment is performed on the N_m capacity curves obtained. For each macroelement it is possible to take into consideration, where necessary, the increase of seismic action that may arise due to the relative stiffness with the nearby macroelements or to the elements which connect the macroelements (flexural stiffness of transversal wall and/or limited horizontal diaphragm stiffness). This option of the procedure is better described below. Proper combination criteria are introduced to define the performance at the scale of the whole asset, possibly excluding some minor macroelements that are considered not relevant for the fulfillment of the examined performance level.

It is worth noting that the subdivision into macroelements needs not to be the same in both direction and, even when the same macroelement has to be analyzed in-plane and out-of-plane, the best model to be used is usually different (e.g. the façade of a church can be modeled by CCLM or SEM when loaded in-plane and by MBM for analyzing its out-of-plane behavior).

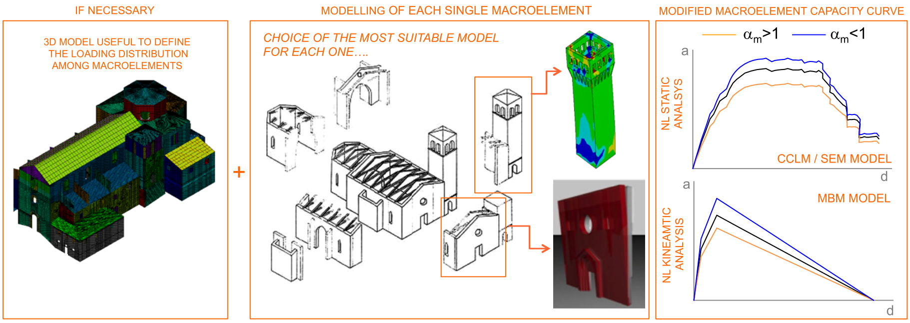


Fig. 15 Subdivision into macroelements, redistribution of seismic actions by a 3D CCLM model and possible effect through the α_m factors

A 3D linear finite element model (Fig. 15) of the whole structure is useful to verify the reliability of the adopted subdivision in macroelements (independence of collapse mechanisms) and to evaluate the redistribution of seismic actions among them (fraction of seismic mass supported by each macroelement). Static analysis with horizontal actions equal to vertical ones can show where first cracks forms and which is the fraction α_m of the total mass m_m that is supported by the m-th macroelement (α_m is obtained by the ratio between the sums of horizontal and vertical constrain reaction at the base of the macroelement). In particular, depending on the connections (in particular the stiffness of horizontal diaphragms), stiffer macroelements attract more actions ($\alpha_m > 1$), while the others have to sustain a lower fraction of seismic forces ($\alpha_m < 1$); however this latter case has to be considered with caution (in doubt it is better to use $\alpha_m = 1$). In order to be on the safe side the assumed mass redistribution must satisfied the following condition:

$$\sum_{m=1}^{N_m} \alpha_m m_m \geq \sum_{m=1}^{N_m} m_m \tag{10}$$

As an alternative, coefficients α_m can be obtained by modal and response spectrum analysis.

The PBA of each macroelement is made using the procedure described in Sect. 7.2. The only difference is that the seismic action that has to be considered for the assessment of the macroelement may be higher or lower compared with what would occur if the macroelement were isolated ($\alpha_m = 1, m = 1, \ldots, N_m$). These coefficients are directly used in the PBA, as they modify the capacity curve. Figure 16 shows that if $\alpha_m > 1$ the acceleration capacity a is reduced (orange lines), while macroelements in which $\alpha_m < 1$ take profit of the redistribution (blue lines).

Once the seismic assessment of each single macroelement has been made, with the evaluation of $IM_{PL,m}$, it is necessary to introduce proper criteria to assess the maximum demand compatible for the whole asset ($IM_{PL,G}$). The simplest conservative method is to assume $IM_{PL,G}$ as the minimum value provided by the assessments made on the whole set of macroelements:

$$IM_{PL,G} = min\left(IM_{PL,m}\right) \quad (m = 1, \ldots, N_m) \tag{11}$$

As an alternative, in order to be consistent with the multiscale approach adopted for the definition of PLs in complex assets described by a global pushover curve, the following procedure is proposed. First of all a weight ρ_m has to be assigned to each macroelement ($\Sigma_m \rho_m = 1$), as a function of its dimension and/or relevance in the building. Then the fragility curve of the performance level kn ($k = 1, \ldots, 4$; $n = U, B, A$) of the whole asset is evaluated as:

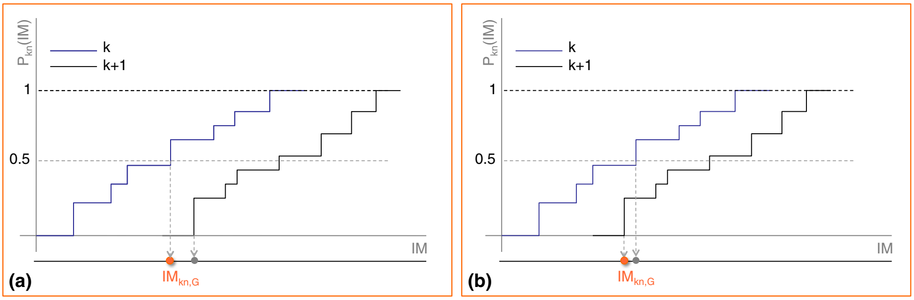


Fig. 16 Evaluation of IM_{PL} in case of assets described by N capacity curves: **a** $IM_{kn,G}$ corresponding to the lower value of IM for which the fragility curve has $P_{kn} \geq 0.5$; **b** $IM_{kn,G}$ corresponding to the value of IM for which the fragility curve of the performance level $(k + 1)$ is greater than 0

$$P_{kn}(IM) = \mathrm{P}\left[\mathrm{PL} \geq \mathrm{kn}|IM\right] = \sum_{m=1}^{N_m} \rho_m \mathrm{H}\left(IM - IM_{kn,m}\right) \tag{12}$$

where H is the Heaviside function (0 if $IM < IM_{PL,m}$; 1 otherwise).

Finally, the value of $IM_{PL,G}$ is obtained as the minimum of the following two conditions: i) the lower value of IM for which the fragility curve has $P_{kn}(IM) \geq 0.5$; ii) the value of IM for which the fragility curve of the performance level $(k + 1)n$ is greater than 0. This can be formally written as follows:

$$IM_{kn,G} = \min \begin{cases} P_{kn}^{-1}(0.5) \\ \min\left(IM_{(k+1)n,m}\right) \quad m = 1, \ldots, N_m \end{cases} \tag{13}$$

where P_{kn}^{-1} is the inverse function of the fragility curve (12).

Figure 16 summarizes the procedure proposed to compute $IM_{PL,G}$ at the scale of the whole asset; in case a) the first condition prevails, whereas in case b) the second one is crucial. A case study of application of this procedure is the Great Mosque of Algiers (Rossi et al. 2014).

7.3 Nonlinear dynamic analyses (IDA, MSA)

Nonlinear dynamic analysis is a more accurate method for the seismic assessment, because it models the dynamic behavior of the structure and does not need the conventional transformation to an equivalent nonlinear single degree of freedom system; in this way the contribution of all modes is implicitly considered, as well as the effect of vertical component of the input motion, sometimes not negligible. However, the higher computational effort and some additional modeling features (e.g. cyclic hysteretic behavior of structural elements, not needed for pushover analysis) make the method feasible only in a limited number of cases and mainly for some classes (for example, Classes F and C or simple buildings belonging to Classes A and B).

The application of an acceleration time history at the base of the structure and the evaluation of its nonlinear dynamic response produce a large amount of results: time histories of nodal displacements, element drifts, local and global energy dissipation. These data must be properly processed in order to assess if a given PL has been attained or not. This is not a simple task and many alternative approaches have been proposed in the past, usually referred to the definition of a global damage index that can be correlated with the PLs. In order to be compatible as much as possible with the multiscale approach defined for the static nonlin-

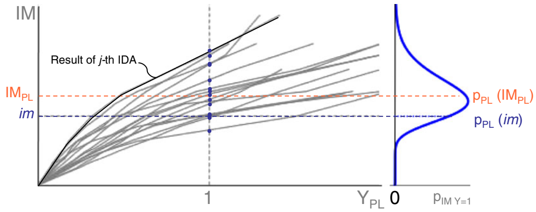


Fig. 17 Evaluation of IMPL from the results of IDA with a sufficient number of records

ear procedure (pushover-CSM), described in the previous section, PERPETUATE guidelines propose to make reference to a scalar variable:

$$Y_{kn,G} = \max\left[\frac{\max(\Lambda_{P,DLk};\Lambda_{S,DLk})}{\Lambda_{P-S}};\ \max\left(\frac{\theta_{w,l}}{\theta_{DLk}}\right);\ \frac{d_{max}}{d_{DLk,G}}\right] \quad (14)$$

where: $d_{DLk,G}$ is the limit displacement threshold that defines DLk at global scale, from a heuristic check on the pushover curve obtained by nonlinear static analyses, and d_{max} is the maximum of absolute values attained during the time history of the representative displacement d (the one used in the pushover curve).

It is assumed that, after processing the results of a nonlinear dynamic analysis, the attainment of $Y_{kn,G} = 1$ indicates the reaching of the examined PL (kn). The results of a series of nonlinear dynamic analyses may be represented by plotting $Y_{kn,G}$ as function of *IM*.

If the PBA is made by the Incremental Dynamic Analyses (IDA) procedure, a set of N_j records is used. The *j-th* record is incrementally scaled through IM until the threshold $Y_{kn,G} = 1$ is reached, and the corresponding $IM_{kn,j}$ $(j = 1, \ldots, N_j)$ is obtained. IDA curves are plotted in Fig. 17. The outcome of the PBA, that is the mean value of maximum IM that is compatible with the fulfillment of the given PL is obtained as:

$$IM_{kn,G} = \frac{1}{N_j}\sum_{j=1}^{N} IM_{kn,j} \quad (15)$$

Another option is the use of the cloud method, that consists in the performing a large number of nonlinear dynamic analyses, by using recorded time histories without any scaling (because the admissibility of scaling records is questioned by some Authors). In this case the probability distribution of $IM_{kn,G}$ can be obtained by MSA, and then the mean value is evaluated.

7.4 Seismic assessment of possible local mechanisms

All the previous issues implicitly refer to the assessment of the building as a whole, the total seismic mass of the structure being involved. However, it is worth pointing out that an exhaustive seismic assessment would require also the verification of possible local mechanisms (mainly out-of-plane ones). The attribute "local" refers to mechanisms that involve only a portion of the building and may not be accurately analyzed by the global or macroelement mechanical model; for this reason they are studied by a different specific local model, by considering only a fraction of the total mass. To this aim, all kinds of models can be used but the MBM approach appears to be the most effective.

In general, these mechanisms have to be considered only if relevant for the PLs of the examined asset. If N_o local mechanisms have been selected for the given PL, the PBA of these mechanisms provides, as in the case of the global asset scale, values of the IM that is compatible with the attainment of the examined PL (kn): $IM_{kn,o}(o = 1, \ldots, N_o)$. The final outcome of the PBA is given by:

$$IM_{kn} = min\left[IM_{kn,G}; \quad IM_{kn,o}(o = 1, \ldots, N_o)\right] \tag{16}$$

where, as afore introduced, $IM_{kn,G}$ is the value obtained at the global scale, according to formulations proposed in Sects. 7.2.4, 7.2.5 and 7.3 (simply named $IM_{PL,G}$ in Eq. 9).

Since local mechanisms usually involve portions located at a higher level of the building (different from the ground floor), it is necessary to adopt proper modified response spectra aimed to take into account the filtering effect provided by the structure. By considering analytical *floor spectra* (Suarez and Singh 1987) and after an in-depth calibration supported by dynamic analyses, the following expressions are proposed in Lagomarsino (2014):

$$S_{dZ}(T, z) = \max\left[S_d(T); \quad \sum_{1}^{N_r} S_{dZ,r}(T, z)\right] \tag{17}$$

where: $S_d(T)$ is the displacement response spectrum of the ground motion; N_r is the number of considered building modes; $S_{dZ,r}(T, z)$ is the contribution of mode r that is given by:

$$S_{dZ,r}(T, z) = \begin{cases} S_d(T_r)\dfrac{\gamma_k|\psi_k(z)|\left(\frac{T}{T_r}\right)^2}{\sqrt{\left(1-\frac{T}{T_r}\right)^2 + \frac{0.05}{[\eta(\xi)\eta(\xi_b)]^2}\frac{T}{T_r}}} & T < T_r \\ S_d(T_r)\dfrac{\eta(\xi)\eta(\xi_b)\gamma_k|\psi_k(z)|\left(\frac{T}{T_r}\right)^2}{\sqrt{\left(1-\frac{T}{T_r}\right)^2 + 0.05\frac{T}{T_r}}} & T_r \le T \le 1.9T_r \\ 3.8\eta(\xi)\eta(\xi_b)\gamma_r\,|\psi_r(z)|\,S_d(T_r) & T > 1.9T_r \end{cases} \tag{18}$$

where: T_r, ψ_r and γ_r are period, modal shape and coefficient of participation of mode r, respectively; $\eta(\xi)$ and $\eta(\xi_b)$ are the damping correction factors of the local mechanisms and the building, respectively.

Figure 18 shows an example of local mechanism in the belfry, which is typical in the case of a bell tower (an asset of Class C) and summarizes the procedure to compute the final value of IM_{PL} that have to consider both the global response and that of local mechanisms. In particular, Fig. 18 depicts the case in which the local mechanism examined is verified with the demand scaled to the $IM_{PL,G}$ value (considering both spectra, that at ground level and the amplified floor one): it means that it is not necessary to proceed to the computation of $IM_{PL,L}$ being surely the lower value that associated to $IM_{PL,G}$.

7.5 Seismic assessment of artistic assets

The relevant immovable artistic assets for which it is necessary to assess some PLs belong to three different classes, as already introduced in Sect. 3:

- Class P—artistic structural elements (e.g. carved stone columns, decorated wooden beams): their performance can be related to the same parameters used for the definition of structural damage levels (drift limits), possibly adopting specific values related to the consequences to the artistic asset;
- Class Q—artifacts strictly connected to structural elements (e.g. frescoes, mosaics, stuccoes): their performance is defined by parameters other than the structural ones, but a direct correlation between them can be established (e.g. in the case of frescoes, a parameter for measuring the damage level could be the maximum width of cracks, which is

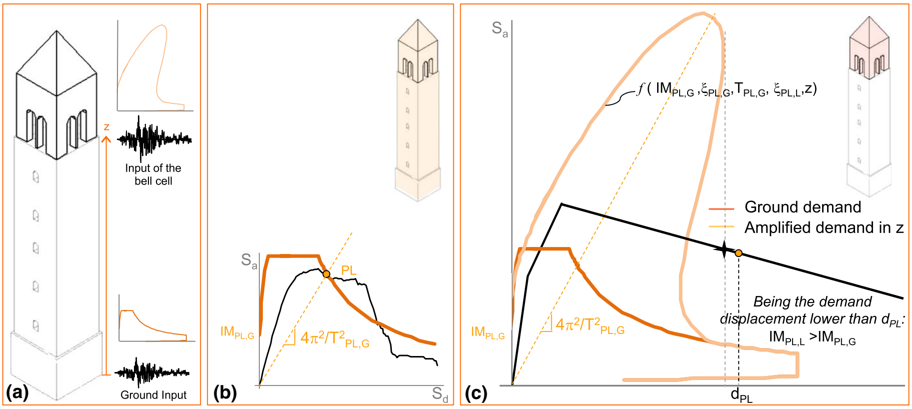


Fig. 18 Example of a local mechanism (the belfry in **a**) for which it is necessary to evaluate the filtering effect: **b** computation of $IM_{PL,G}$ (from ADRS at ground level); computation of $IM_{PL,L}$ (from amplified ADRS)

correlated to the drift of the panel where the decorated plaster is applied—see Calderini et al. 2014);

- Class R—artistic assets that are independent elements (e.g. sculptures, balconies, pinnacles, merlons): usually these assets can be modeled by MBM and the PBA must consider amplified ADRS (introduced in the previous section), because their position is usually at the upper levels of the building.

It is worth noting that for the first two classes, PLs are identified through a local check in the element or macroelement where the artistic asset is placed. The point of the pushover curve in which the PL is attained is identified and the PBA is made by using the global pushover curve of the architectural asset (Fig. 1) or that of the macroelement (in the case of assets described by more than one capacity curve). An illustration of the definition of PLs on the global pushover curve in the case of artistic assets of class Q is presented in Rossi et al. (2014).

8 Rehabilitation decisions

By means of the hazard curve, the calculated values of IM compatible with the required PLs (IM_{PL}) are converted into corresponding return periods $T_{R,PL}$ in order to be compared with $\bar{T}_{R,PL}$, the return period of the target earthquake level requested for the given PL.

A safety index $I_{S,PL} = T_{R,PL}/\bar{T}_{R,PL}$ can be defined, being greater than 1 when the safety requirements are fully satisfied. It allows to draw a priority list of interventions, in case of assessment of a group of buildings.

Another possible interpretation is through the evaluation of the nominal life V_N of the asset, which is defined as the number of years in which the building can be used and the architectural and artistic assets can be considered preserved from earthquake risk, assumed that it is subject to regular maintenance (Recommendations P.C.M. 2011). Since hazard levels are usually defined for probabilities of exceedance in 50 years, the nominal life is given by:

$$V_{N,PL} = 50\frac{T_{R,PL}}{\bar{T}_{R,PL}} = 50 I_{S,PL} \tag{19}$$

Thus, it may be assumed that, if $V_{N,PL} > 50$ years, the seismic performance of the architectural asset is adequate, while, if $V_{N,PL} < 50$ years, rehabilitation decisions have to be taken.

The nominal life $V_{N,PL}$ is a useful parameter to quantify the time within which preventive actions have to be implemented. This approach is correct if Building conservation (*B*) and Artistic assets conservation (*A*) targets of performance are considered, because the accepted safety level refer to a probability of occurrence in a long window of time. On the contrary, as far as Use and Human life (*U*) performance level are concerned, in particular *3U* (Life Safety), it is evident that the accepted probability of occurrence refers to a short time window (e.g. annual rate of exceedance), because it is related to the presence of people as occupants of the buildings.

However, it is worth noting that the use of $V_{N,PL}$ is correct from a conceptual point of view only if a time-dependent hazard is available. In the most common case of a Poisson hazard model, the definition of $V_{N,PL}$ represents only a rational approach to face the problem of finding a balance between safety and conservation requirements. In fact, in the case of a PBA made on a group of cultural heritage assets it is possible to compare the safety levels with reference to the forthcoming years, planning and optimizing the preventive actions.

In case the safety verification highlights the need of improving the seismic capacity of the building, different rehabilitation alternatives may be considered. First of all it is advisable to prevent from possible local mechanisms, in particular if the PL is attained due to one of them; in these cases, strengthening interventions can be realized quite easily, are less invasive and do not modify so much the global behavior. The results of nonlinear analyses can help to single out the weaker elements and detect the irregular behaviors (torsional effects, irregularity in elevation); seismic preventive interventions should be focused to the aim of adjusting them, rather than proceeding to a generalized strengthening of all elements.

The design of strengthening interventions is not the only possible choice. Conservation without interventions may be also considered, if strengthening actions would be too invasive. In this case, the nominal life of the building is considered and further decisions are postponed. Another alternative is the revision of safety requirements, which in practice, means the change of use of the building. Finally, the building may be monitored and models upgraded. Also in this case the choice is postponed to the future, when more accurate and validated tools will be available for sure, due to the improvements of applied research in the field.

9 Possible improvement of the assessment by a probabilistic approach

As also in the case with the aforementioned standards for seismic assessment of existing buildings (ASCE/SEI 41–13 2014; EN 1998–3 2005), the general framework of the PERPETUATE guidelines for cultural heritage assets is semi-probabilistic. Seismic occurrence is defined by a probabilistic hazard curve, but the assessment does not provide the probability of exceeding the required performance levels; the outcome is the seismic intensity measure that induces the attainment of each performance level and the related mean return period. Moreover, the seismic capacity of the building is assumed as deterministic, hence neglecting the effect of both epistemic and aleatory uncertainties, such as: i) the reliability of mechanical models; ii) the incomplete knowledge of the building; iii) the spatial and statistical dispersion of material parameters; iv) the variability of seismic input motion not explicitly taken into account by the assumed IM. These uncertainties are only partially considered in the preliminary sensitivity analysis and included through the CF (Sect. 6).

However, the framework of the PERPETUATE guidelines allows improving the verification towards a probabilistic assessment, without a significant increase in the computational effort, by adopting the SAC-FEMA approach (Cornell et al. 2002), recently applied to existing r.c. buildings through the use of pushover analysis (Fajfar and Dolšek 2012). The SAC-FEMA approach provides an approximation of the probability of exceedance (or mean annual frequency) of a given performance level by the following expression:

$$P_{PL} = \tilde{\lambda}\,(IM_{PL})\, C_{H,PL} C_{V,PL} \tag{20}$$

where: $\tilde{\lambda}\,(IM_{PL})$ is the median value of the hazard function at IM_{PL}, the value of IM that causes the selected performance level according to the procedure proposed in PERPETUATE guidelines (which is here considered as median value), $C_{H,PL}$ is a factor that transforms between median and mean hazard values:

$$C_{H,PL} = e^{\frac{1}{2}\beta^2_{H,PL}} \tag{21}$$

where $\beta_{H,PL}$ is the dispersion of the hazard curve at IM_{PL}, and $C_{V,PL}$ is a factor that accounts for the dispersion in the evaluation of IM_{PL} due to the uncertainties in the capacity and the characteristics of seismic demand not included in the hazard curve (e.g. the dispersion of the ADRS shape due to the record-to-record variability):

$$C_{V,PL} = e^{\frac{1}{2}k^2\left(\beta^2_{C,PL}+\beta^2_{D,PL}\right)} \tag{22}$$

where $\beta_{C,PL}$ and $\beta_{D,PL}$ are the above mentioned dispersions related to the capacity and the demand, q is a parameter of the hazard curve, idealized in the form:

$$\tilde{\lambda}\,(IM) = q_0\ IM^{-k} \tag{23}$$

The transition to a fully probabilistic assessment is then basically limited to the evaluation of the dispersions $\beta_{C,PL}$ and $\beta_{D,PL}$. After the sensitivity analysis (see Sect. 6), the parameters that mainly influence the seismic behavior are singled out and must be defined as aleatory variables; then, the dispersion $\beta_{C,PL}$ can be obtained with the response surface technique, through the results of a set of deterministic models defined by a complete factorial combination at two levels (Lagomarsino and Cattari 2014). Similarly, $\beta_{D,PL}$ is obtained considering the uncertainties in the seismic input as modeled by ADRS for different confidence levels, in case of static nonlinear analysis, or directly by the record-to-record variability of the selected accelerograms.

The introduction of a probabilistic approach is useful also to refine the correlation among DLs and PLs. To this aim, fragility functions can be defined in terms of the displacement d of the capacity curve, by considering the values of the displacement compatible with each DL (d_{DL}) and the related dispersion:

$$P_{DLK}(d) = P\,(DL \geq DLk|d) = P\,(d_{DL} < d) = \Phi\left(\frac{log\left(\frac{d}{d_{DLK}}\right)}{\beta_{DLK}}\right) \tag{24}$$

In this case, the dispersion β_{DLk} is related to the displacement of the equivalent SDOF system and can be evaluated through a procedure based on the response surface technique, similar to the one previously mentioned for the dispersion of the IM. This approach is described also in Annex A6 of a recent draft document of recommendations, issued by CNR, Italian National Research Council (CNR-DT212 2013).

A refined definition of acceptance criteria for a given performance level PL (or kn, with $k = 1, \ldots, 4$) is then possible, referring to the probability of having a proper combination

Table 3 Acceptance criteria defined in PERPETUATE guidelines

Performance level (PL – kn)	Acceptance criteria			
	n = B—Building conservation targets		n = U—Use and human life targets	
	Correlation with DL	Limit value (%)	Correlation with DL	Limit value (%)
k = 2	–	–	$0.4\ P_{DL3} + P_{DL4} + P_{DL5}$	1
k = 3	P_{DL5}	3	$0.3\ P_{DL5}$	3
k = 4	P_{DL5}	15	–	–

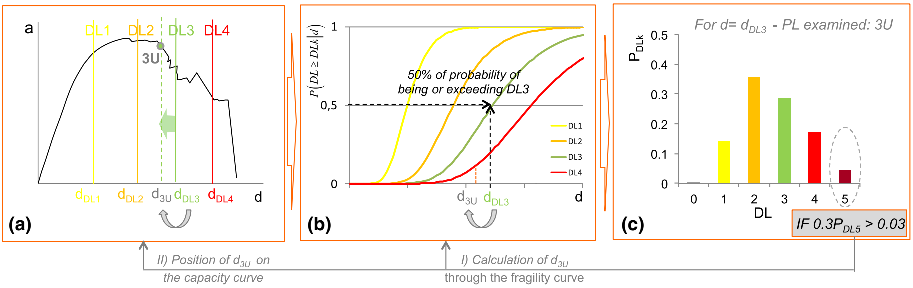


Fig. 19 Probabilistic approach through the use of fragility curves and acceptance criteria: **a** DLs position from the multiscale approach and possible refinement of PLs position after steps **b** and **c**; **b** fragility curve (from the computation of β_{DLk}); **c** computation of the probability distribution for a given d_{DLk} and check for the acceptance criterion adopted (PL = *3U*)

of damage levels probabilities P_{DLk}, given by the above introduced fragility curves. Indeed, since on the pushover curve only the first four damage states are defined, once the overall damage distribution is obtained, it is then possible to split the probability of DS4 into a fraction of DS4 and DS5 (Lagomarsino and Giovinazzi 2006). In particular, the latter step is very useful to refine the assessment of Use and Human Life requirements, since in many cases the correlation laws for computing casualties are related to DS5. Table 3 reports possible criteria derived from correlations between DLs and PLs (Coburn and Spence 2002; Dolce et al. 2006; Goretti and Pasquale 2004; Di Pasquale et al. 1998, 2005; Bramerini et al. 1995), obtained through robust statistical analyses of post-earthquake data. For example *2U* (Immediate occupancy) is related to a limited percentage of DS3/4/5, while *3U* (Life safety) is fulfilled if the probability of having severely injured people, which are related to a percentage of people living in buildings which suffer DS5, is sufficiently low. Through the use of fragility curves it is then possible to define the corresponding point on the pushover curve that satisfies such percentages (Fig. 19). Usually the simplified definition of d_{PL} (or d_{kn}) as coincident to d_{DLk} is acceptable but, if fragility curves are available and for a given PL the criterion is not satisfied, the displacement d_{PL} have to be brought back (Fig. 19). This occurs when DLs are very close one to the other (low ductility) and/or in presence of significant uncertainties in the assessment.

10 Conclusions

The PERPETUATE project has proposed guidelines for the performance-based assessment of cultural heritage masonry buildings and contained immovable artistic assets.

Assuming that preservation of monumental structures requires a detailed knowledge of the seismic behavior in order to minimize the interventions, for the sake of conservation, accurate nonlinear models and the displacement-based approach have been adopted in the assessment procedure.

The general framework of the guidelines is compatible to widely known and used procedures for the assessment of existing buildings (ATC/SEI 41-06, EN 1998–3 2005). However, distinctive features of monumental structures have required making reference to different modeling strategies, both in terms of mechanical approaches (nonlinear finite elements, distinct elements, equivalent frame models, limit analysis of rigid blocks) and scale of analysis (global modeling of the building, subdivision into macroelements, analysis of local mechanisms). Both static pushover and incremental nonlinear analyses are considered.

Even if more research is needed, the main aim of the paper is to give a clear picture of the whole seismic PBA procedure, which is original in that it has framed the problem of verification of monumental masonry buildings within a quantitative approach, compatible with the one adopted for other ordinary and strategic buildings. The results offered by this procedure are also useful to compare the level of risk among various cultural heritage assets that are present in a region, with the aim of planning mitigation measures.

Moreover, some contributions here presented are original: i) the multiscale approach for the definition of PLs on complex and irregular masonry buildings (Sect. 7.2.2); ii) the assessment of a monumental building by subdivision into macroelements and the consequent evaluation of the fragility function of the whole asset (Sect. 7.2.5); iii) the use of the response surface method for the probabilistic-based refinement of the PLs (Sect. 9). Specific insights and more details on some steps of the procedure are provided in other papers of this Special Issue of the Bulletin of Earthquake Engineering, as well as some applications to relevant case studies.

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