PERPETUATE

PERformance-based aPproach to Earthquake proTection of cUlturAl heriTage in European and mediterranean countries

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European Guidelines for the seismic preservation of cultural heritage assets

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SUMMARY

In this deliverable, the *European Guidelines for the seismic preservation of cultural heritage assets* are presented. Starting from the idea that a reliable assessment procedure can be the main tool to respect the principle of "minimum intervention" under the constraint of structural safety, this document presents the full methodological path for the assessment of cultural heritage assets proposed in PERPETUATE project.

In particular, target performance levels are properly defined by PERPETUATE for cultural heritage assets, which consider not only the use and safety of people but also the conservation of the architectonic and artistic value of the monument. The displacement-based approach for vulnerability assessment of cultural heritage assets and design of interventions is adopted as standard method of analysis. Thus, nonlinear analyses, both static and dynamic, are adopted as the key tool for evaluating the seismic behaviour of masonry structures after damage occurs, till to near collapse conditions. A classification of architectonic and artistic assets is proposed and typical damage modes are described in order to address the possible modelling strategies. Four alternative types of models have been identified and developed with the aim of analysing the wide variety of historical masonry constructions.

Summarized in an oriented practice tool, Guidelines outline the procedure proposed based on three main steps (focusing the attention on the scale of single asset). In the first one, the building is known and the seismic input is defined in accordance with specific safety and conservation requirements. In the second one, the seismic response of the asset is described through mechanical model and its capability to satisfy the defined requirements is assessed. Finally, in the third step, rehabilitation decisions are taken and, if necessary, the second step is repeated for the design of strengthening interventions Then, the complete set of deliverables constitute the main reference to deepen the steps and seismic assessment criteria summarized in the Guidelines.

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1. Preface

The damage assessment to cultural heritage assets after recent earthquakes showed the high vulnerability of some types of historical structures (churches, palaces, towers …). Earthquakes also proved that strengthening interventions adopted in the last decades are invasive, not effective and even increase the vulnerability. Thus, there is an urgent need for promoting a new and really effective strategy for the risk mitigation of cultural heritage.

The preservation of cultural heritage assets must guarantee their capacity of lasting over time against decay, natural hazards and extreme events, without losing their authenticity and usability. This means that the need of guaranteeing an “acceptable level” of structural safety for buildings occupants should be always related to the principle of “minimum intervention” on the building itself. The definition of “acceptable” safety levels, as well as the concept of “safety”, still represents an open issue for monumental buildings.

Furthermore, it should be considered that the intangible value of these buildings depends, besides their social and historical meaning, on both architectural and artistic factors. Thus, their risk assessment is a challenge regarding not only structural and architectural components, but also movable (paintings, statues, libraries…) and unmovable (frescos, stucco-works, pinnacles, battlements, banisters, balconies) artistic assets contained in it.

PERPETUATE guidelines propose a methodology for the assessment of seismic risk to cultural heritage assets and design of interventions, based on the following principles:

- The protection of cultural heritage needs an improvement in methods of analysis and assessment procedures, rather than the development of new intervention techniques. A reliable assessment procedure is the main tool to respect the principle of “minimum intervention” under the constraint of structural safety.
- The displacement-based approach for vulnerability assessment of cultural heritage assets and design of interventions is adopted as standard method of analysis. The mechanical models available for the analysis of ancient masonry buildings or elements usually consider a verification approach in terms of forces, because in the past strengthening techniques were based on the concept of increasing stiffness and strength. This approach is correct in static conditions, but earthquake is a dynamic phenomenon that induces deformations and dynamic amplification. Usually much more the construction is stiffer much higher are the equivalent static actions, which is subjected to: thus, flexibility is a first positive characteristic for a good seismic behaviour. Moreover, since inertial actions are proportional to the weight of the construction, the lightness is a second fundamental aspect. Finally, in the case of rare destructive earthquakes, it is impossible to bear the seismic actions without significant damage. Hence, the building must be ductile enough and be able to sustain vertical loads without collapse, even if large horizontal displacements are induced by the earthquake.
- Despite the complex nature of heritage buildings and artistic assets, the DBA (Displacement Based Assessment) calls for nonlinear models. Nonlinear static (pushover) analyses are considered as the main tool for the application of the assessment procedure. Nonlinear dynamic analyses are considered as an alternative tool only for certain types of assets.

- If, by reliable methods of analysis and assessment procedures, it is demonstrated that the monument is not safe, then its retrofitting is unavoidable, first of all in order to preserve its life along the time and also for the safety of occupants. An effective improvement does not necessarily need the development of new materials and intervention techniques. In many cases, traditional techniques represent a sustainable, effective, cost-efficient and reversible solution, whose effectiveness is proved by the time and also by the results of new methods of analysis.
- A reliable assessment procedure of heritage buildings requires that both architectonic and artistic assets contained in them are considered.

The procedure here described is oriented to the assessment of single cultural assets. PERPETUATE has extended the concept of “single cultural asset” also to the scale of the historical centre, intended as a complex asset that, although made of many buildings, has a cultural relevance in its systemic unity. This Guidelines are related to single buildings but information on the assessment at urban scale are contained in Deliverables D27 and D40, where the method has been applied to the Casbah of Algiers and the historical centre of Ljubljana (Slovenia), as well as to single asset case studies.

2. Seismic Performance-Based Assessment of architectonic and artistic assets

Seismic performance-based assessment (PBA) of an existing building checks if the construction is able to fulfil some selected performance levels (PLs) in case of occurrence of properly defined earthquake hazard levels, in terms of annual rate of exceedance (or return period T_R).

Target performance levels are properly defined by PERPETUATE for cultural heritage assets, which consider not only the use and safety of people but also the conservation of the architectonic and artistic value of the monument.

Nonlinear analyses, both static and dynamic, are necessary for evaluating the seismic behaviour of masonry structures after damage occurs, till to near collapse conditions. A classification of architectonic and artistic assets is proposed and typical damage modes are described in order to address the possible modelling strategies. Four alternative types of models have been identified and developed with the aim of analysing the wide variety of historical masonry constructions.

Nonlinear static (pushover) analysis is considered the main tool for the PBA and detailed acceptance criteria are proposed for the identification of PLs on the pushover curve, related to the different targets of performance (Figure 1). Nonlinear dynamic analyses represent an alternative tool, suitable for some types of architectonic assets and useful in some specific conditions. Rarely it is necessary to restrict to linear elastic analysis, due to the complexity of the model if the building is huge; hints are given for the definition of a simplified capacity curve and the PLs, in order to perform also in this case a displacement-based assessment.

The seismic input is represented by an Acceleration-Displacement Response Spectrum (ADRS), completely defined for the specific site of the building under investigation as a function of one parameter assumed as Intensity Measure (IM) of the earthquake. Possible IMs are: peak ground acceleration, spectral acceleration for a given period, maximum spectral displacement, Arias intensity, Housner intensity. A Probabilistic Seismic Hazard Analysis (PSHA) is necessary in order to evaluate the annual probability of occurrence (or the return period) of earthquakes of different values of IM (hazard curve). In some cases a Vector-Valued PSHA may be used, in order to better describe the characteristics of the seismic input (hazard surface).

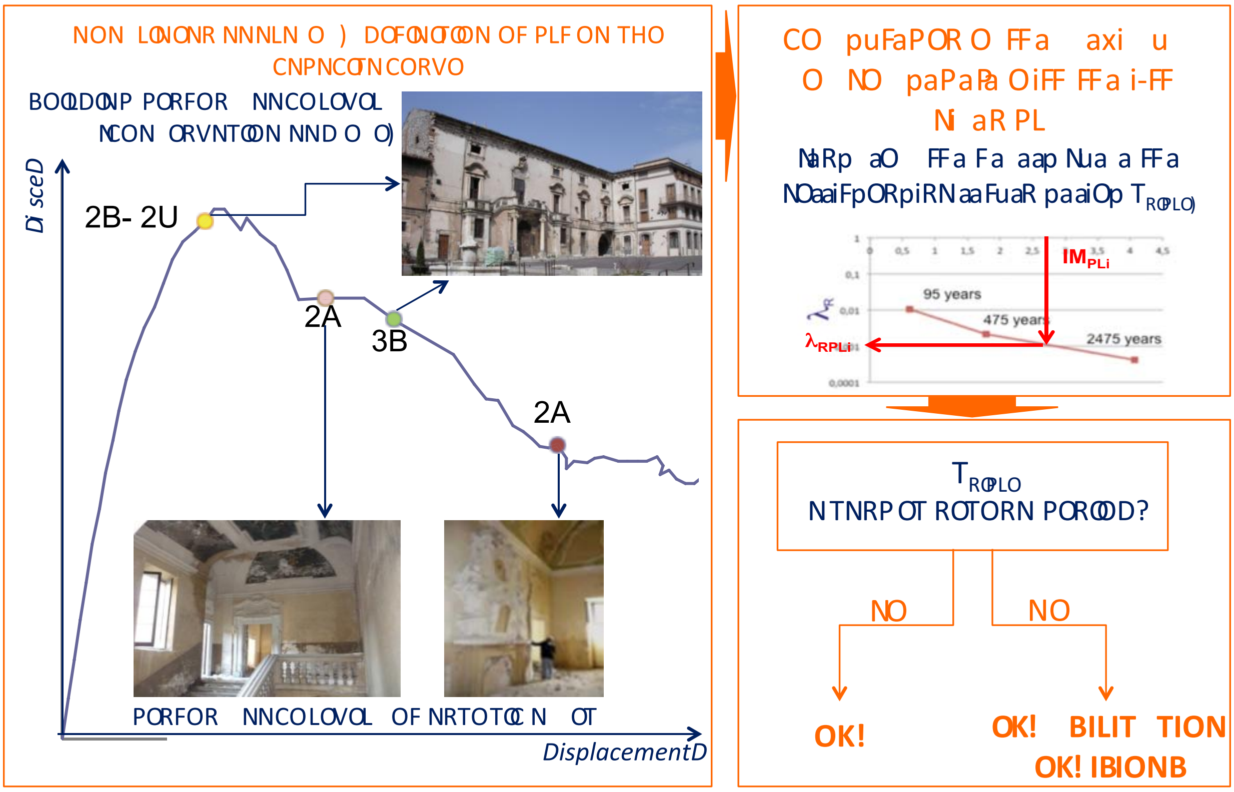

Figure 1. The performance-based assessment of architectonic and artistic assets: pushover curve, performance levels and return periods of the seismic input compatible with target performance.

The outcome of the assessment is the maximum IM compatible with the fulfilment of each performance level that has to be considered for the asset; through the hazard curve, it is possible to evaluate the annual rate of exceedance λ (or the earthquake return period $T_{R,PL} \cong 1/\lambda$) correspondent to this performance. These values may be compared with the target return period assumed for the assessment, as a function of the characteristics of the asset.

The format of the assessment proposed by PERPETUATE guidelines is deterministic, except for the occurrence of the earthquake, as well as in all codes and recommendations worldwide adopted at present. However, it is well know many uncertainties, aleatory and epistemic, affect the assessment of an existing masonry building, with reference to: a) the relevant characteristics of seismic input (duration, frequency content, etc.); b) the reliability of mechanical models; c) the material parameters; d) the incomplete knowledge of the construction. PERPETUATE takes into account of probabilistic aspects in some steps of the procedure: acceptance criteria for the definition of PLs; sensitivity analysis for drawing the protocol of in-situ investigations. However, a simplified fully probabilistic assessment is possible, starting from the outcome of PERPETUATE, through the SAC-FEMA approach (Cornell et al., 2002) applied to existing buildings (Fajfar and Dolšek, 2012), which gives the annual rate of exceedance the performance level λ_{PL}, once the dispersion measures for the aleatory and epistemic uncertainties in displacement demand and capacity have been estimated.

A full methodological path for the assessment of cultural heritage assets is proposed, which is based on three main steps (Figure 2). The first one includes: 1) classification of the architectonic asset and contained artistic assets; 2) definition of performance limit states (specific for the cultural heritage assets); 3) evaluation of seismic hazard and soil-foundation interaction; 4) construction knowledge (non-destructive testing, material parameters, structural identification). The second step is related to: 1) the definition of structural models for the seismic analysis of the masonry building and the contained artistic assets; 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.

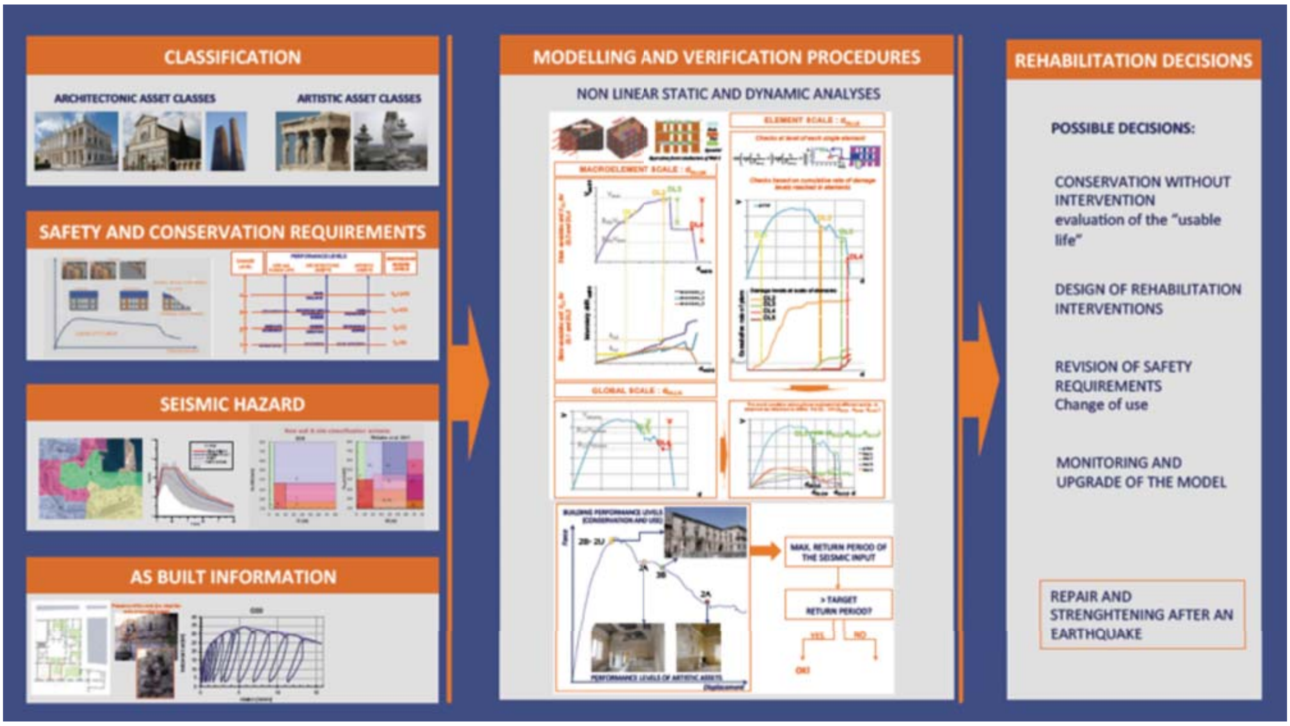

Figure 2. PERPETUATE PBA procedure.

3. Classification of the architectonic asset and contained artistic assets

The seismic assessment of a cultural heritage asset requires a deep preliminary knowledge of the construction from the historical point of view, in order to single out the main elements of authenticity and architectural value, as well as the transformations occurred along the past centuries. It is necessary to be aware of the relevance of the different parts of the construction, establishing a hierarchy among architectural elements with the aim of choosing, among possible alternatives, the less invasive strengthening solution.

PERPETUATE considers a classification of architectonic assets (Table 1), which is useful for addressing the choice of proper mechanical models to be adopted for the assessment, which are classified with reference to the modelling scale (masonry material or structural elements) and the type of discretization (continuous or discrete) (Figure 3):

- CCLM (Continuous Constitutive Law Models): finite element modelling with phenomenological or micromechanical homogenized constitutive laws;
- SEM (Structural Elements Models): equivalent frame modelling by discretization in terms of piers, spandrels and other linear and nonlinear elements;
- DIM (Discrete Interface Models): discrete modelling of blocks and interfaces;
- MBM (Macro-Blocks Models): application of limit analysis (safe and unsafe theorem) to a predefined collapse mechanism of rigid blocks.

Table 1. Classes of buildings and related types of models.

Architectonic asset class		Model type			
		CCLM	SEM	DIM	MBM
A	Assets subjected to prevailing in-plane damage Palaces, castles, religious houses, caravansaries, collective buildings		Global		Local
B	Assets subjected to prevailing out-of-plane damage 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				
G	Built systems subjected to complex damage Historical centres		Global		Local

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

Standard Possible Rare

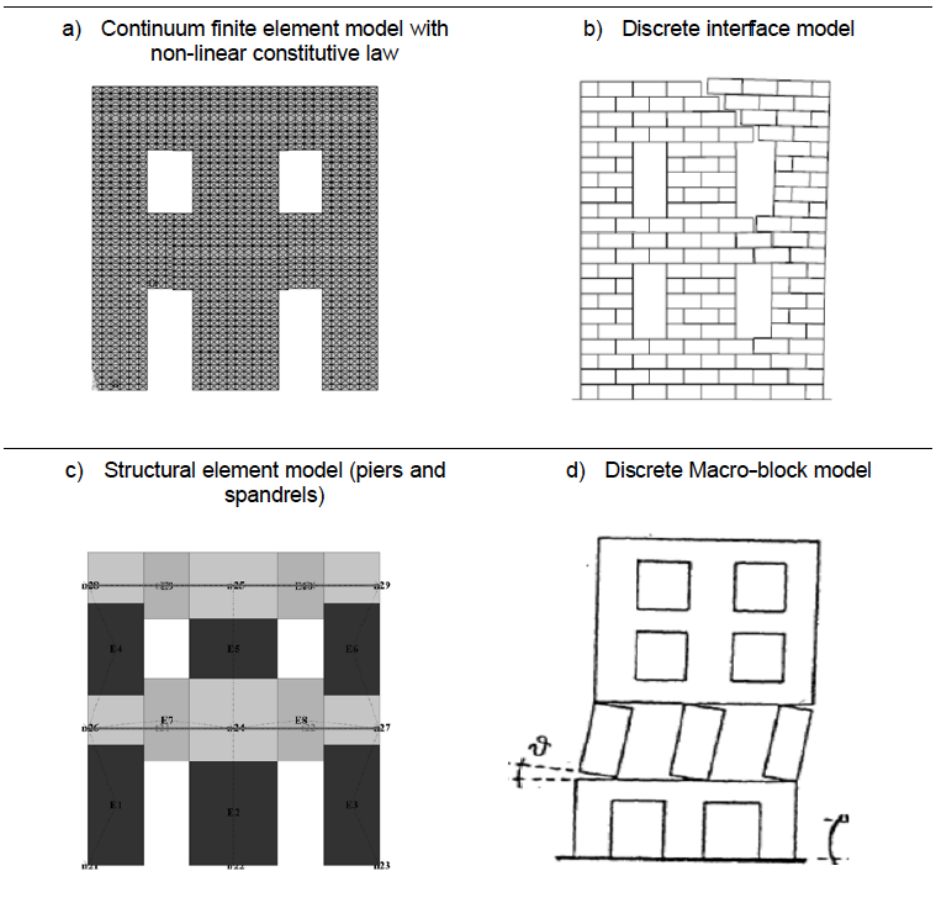

Figure 3. Classification of modelling criteria.

The first aim of the seismic performance-based assessment is to evaluate if the building as a whole is able to fulfil some selected performance levels in case of occurrence of properly defined earthquake hazard levels, in terms of annual probability of occurrence (or return period T_R). Some types of assets (made by a single element or characterised by a so called “box-type” behaviour) can be studied by a global 3D model, while in other cases (complex assets made by macroelements that behaves quite independently) it is necessary to develop more than one model, even of different types (Figure 4). In the latter case the results of the seismic assessment in each macroelement must be properly blended.

Moreover, it is necessary to identify the possibility of suffering local seismic mechanisms, usually involving out-of-plane collapse of small masonry portions. These mechanisms have to be studied with proper local models when the global one is not able to consider them properly. The selection of the mechanisms that may be significant can preliminarily identified in this step but a clear definition is possible after a detailed survey of the constructive details of the building (step 4, §5).

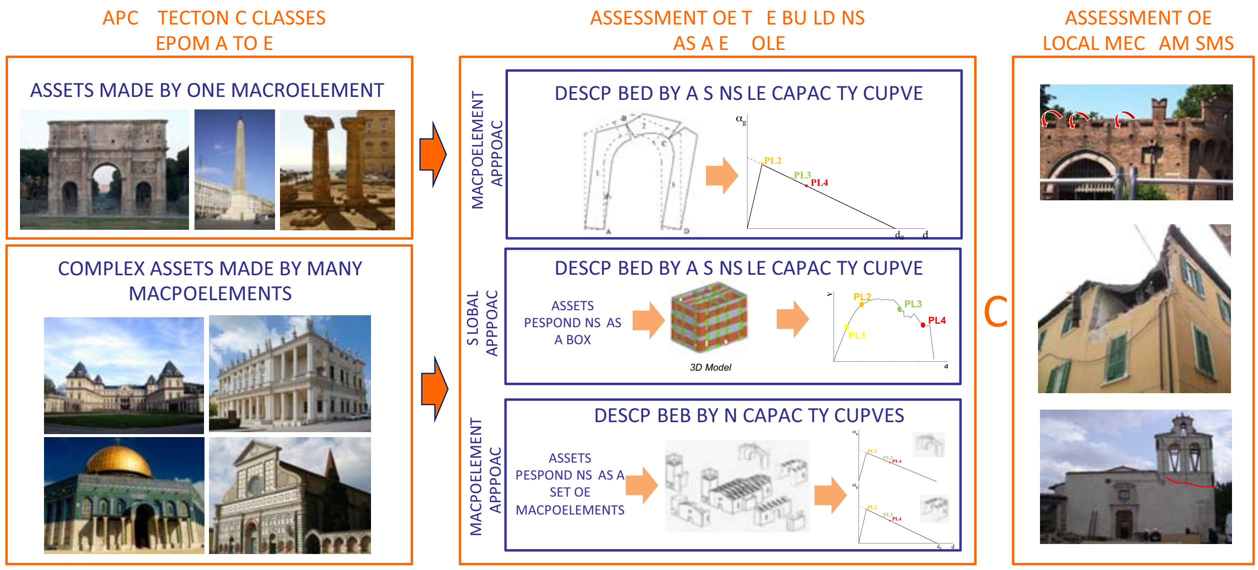

Figure 4. Models for the seismic assessment at global and local scale.

The relevant immovable artistic assets have to be identified and located in the building, establishing a direct relation between each one of them and the structural response of the architectonic asset (Table 2). The assessment in the case of artistic assets of Class P or Q is strictly related with the analysis of the architectonic asset, because they are strictly connected to a structural element or they are a structural element by themselves; on the contrary, in case of Class R, the artistic asset has a own seismic response, which in any case depends on the dynamic input transmitted by the architectonic element on which it is placed. The modelling strategies are the same already introduced and for each Class a standard method is suggested, but other methods may be used, even if rarely.

Table 2. Classes of artistic assets and related modelling strategies.

Class		Str. Relation		Model type			
		Dep.	Ind.	CCLM	SEM	DIM	MBM
P	Artistic assets which are structural elements by themselves Caryatid, carved stone columns, decorated wooden beams…	X					
Q	Artistic assets which are not structural elements (strictly connected to structural elements) Carved stone plates, frescos, mosaics, stuccoes, decorated tiles,…	X					
R	Artistic assets which are not structural elements (with own seismic response) Altars, sculptures, pulpits, Balconies, shelves, gargoyles, bells …		X				

Standard Possible Rare

It is worth noting that only artistic assets that are really relevant for the conservation, on the base of their value, should be included in the seismic assessment. In fact, the need of preservation of an artistic asset can influence significantly the strengthening strategies, as specific techniques (even quite invasive) could be advisable at local scale.

Finally, the use of the building must be known for the seismic assessment. The function can be private, public or even strategic; the activities can determine a rare, frequent or continuous occupation, in some cases with the possibility of crowding.

The aim of the seismic assessment may be different, depending on the current conditions of the architectonic asset:

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

The results achieved in this first step of the seismic assessment are reported in Table 3.

Table 3. Results of step 1: classification of the architectonic asset and contained artistic assets

Outcome		Parameters	Deliverables
1	Classification of the architectonic asset and of its cultural value	cultural value	D4
2	Selection of the type of mechanical model (global or by macroelements) and the type of analysis (nonlinear static or dynamic, linear)	SEM/CCLM MBM/DIM pushover / dynamic	D7 D26
3	Identification of possible local mechanism, to be verified besides of the global seismic response (to be better defined in the as-built information phase)	MBM/DIM	D26
4	Identification of artistic assets, their classification and selection of the proper modelling procedure	Class P/Q/R models	D4 D23

4. Safety and conservation requirements

Displacement-based assessment of existing buildings usually refers to target performance levels, related to the damage in structural and non-structural elements, as well as to the use and safety of occupants (e.g. Immediate Occupancy, Life Safety, Collapse Prevention).

The preservation of cultural heritage assets requires taking into account also conservation objectives, thus PERPETUATE proposes to consider three different targets of performance:

- U - use and human life;
- B - building conservation;
- A - artistic asset conservation.

For each one of these targets, up to four performance levels are defined, which are correlated to five damage levels (DL), defined from the observational approach typical of macroseismic damage assessment (Figure 5): 1) slight; 2) moderate; 3) heavy; 4) very heavy; 5) collapse.

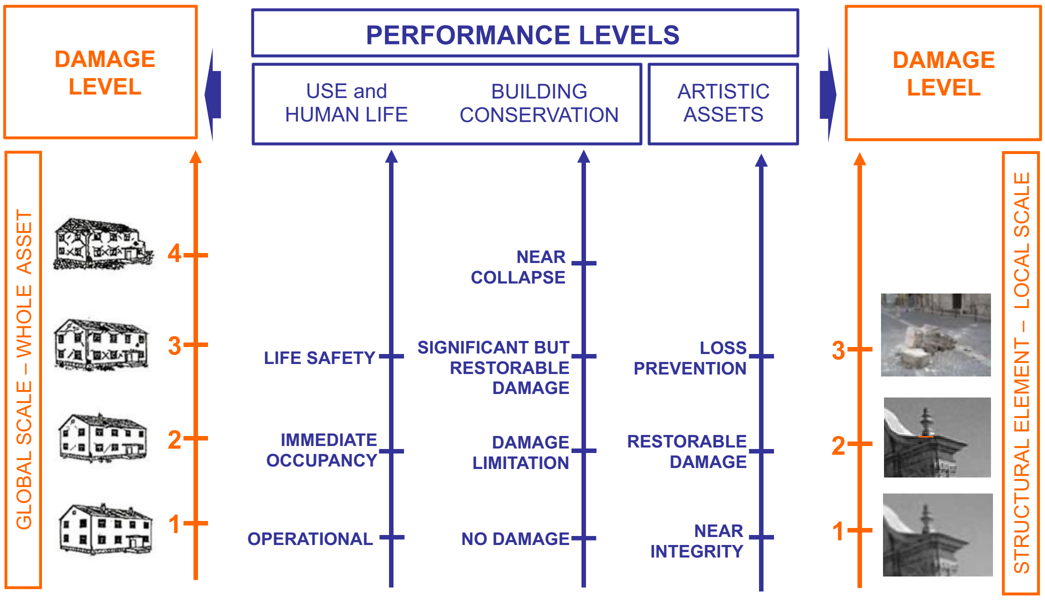

Figure 5. PERPETUATE performance levels are related to damage levels.

The displacement-based assessment procedure defines the damage levels on the pushover curve by proper acceptance criteria, related to the structural response of the model, and establish a correlation with the different performance levels. PERPETUATE proposes a multi-criteria approach that defines the damage levels by considering (e.g. in the case of assets of Class A) the behaviour of single elements (piers and spandrels), macroelements (masonry walls and horizontal diaphragms) and of the entire building (global pushover curve); this topic is addressed on step 5 (§7).

The acceptance criteria assume a direct correlation between DLs and PLs ($d_{PLi}=d_{DLi}$, where d is the displacement in the pushover curve). For example, this means that “life safety” performance level (3U) is fulfilled is the heavy damage threshold is not overcome. Actually, casualties are usually related to the very heavy damage level but, from a probabilistic point of view, the occurrence of very heavy damage in correspondence with a deterministic evaluation of a displacement demand equal to d_{DL3} is not excluded.

A more refined definition of acceptance criteria for a given PLk (k=1÷4) refer to the probability of having a proper combination of the following damage levels P_{DLi} (i=k+1÷5), which can be obtained through fragility curves (after having estimated a proper dispersion measure for uncertainties in displacement demand and capacity). Table 4 reports the proposed criteria; if for a given PLk the criterion is not satisfied in the corresponding displacement d_{DLk}, the displacement d_{PLk} have to be brought forward (Figure 6).

Table 4. Acceptance criteria defined in PERPETUATE procedure.

Performance level (PL)	Acceptance criteria			
	B - Building Conservation Targets		U - Use and Human Life Targets	
	Correlation with damage levels (DL)	Limit value	Correlation with damage levels (DL)	Limit value
2	-	-	0,4 P_{DL3}+ P_{DL4}+ P_{DL5}	1%
3	P_{DL5}	3%	0,3 P_{DL5}	3%
4	P_{DL5}	15%	-	-

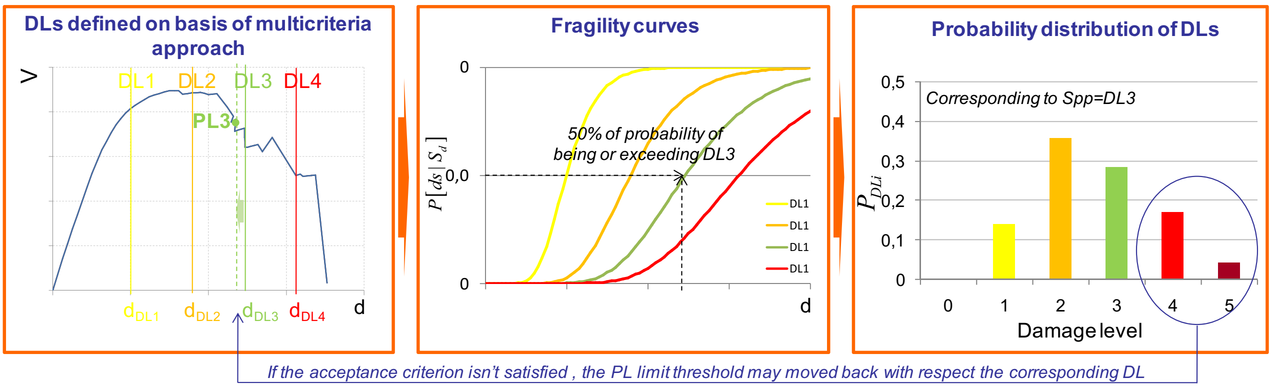

Figure 6. Probabilistic approach through fragility curves adopted to define acceptance criteria.

In this step of the PBA it is necessary to select which performance levels have to be considered for the assessment and the related target earthquake hazard levels. For each target of performance, a primary and a secondary PL are identified (Table 5), as well as the related earthquake hazard levels, expressed in terms of return period ($T_{R,PLi}$). The return periods may be modified by importance coefficients (γ_k, k=U,B,A) that take into account the conditions of use (public, strategic) and the architectonic and artistic value of the examined building. The verification of the primary ones is mandatory, while that of secondary ones is required only for relevant situations (γ_k>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 (PLkA) have to be considered only if relevant artistic assets are present in the building. In this case the damage levels, and the corresponding acceptance criteria, refer only to the element (or macroelement) where the artistic assets is located. The position of the related displacement on the pushover curve may differ significantly from the corresponding ones of Use and Human life and Building conservation (see Figure 1).

Table 5. Primary (red) and secondary (orange) performance levels and related target return periods.

	U - Use and Human life		B - Building conservation		A - Artistic assets	
$T_{R,PLi}$ / γ_k (k=U,B,A)	Immediate Occupancy	Life Safety	Significant but restorable damage	Near Collapse	Restorable Damage	Loss Prevention
72/ γ_k	2U				2A	
475/ γ_k		3U	3B			3A
2475/ γ_k				4B		

The results achieved in this second step and useful for the prosecution of the seismic assessment are reported in Table 6.

Table 6. Results of step 2: target performance levels and earthquake hazard levels.

Outcome		Parameters	Deliverables
1	List of target performance levels to be considered in the PBA, related to: U) use and human life; B) building conservation; A) artistic asset conservation	PLik	D4
2	Importance coefficients, related to: U) use and human life; B) building conservation; A) artistic asset conservation	γ_k k=U,B,A	D4
3	Target earthquake hazard levels	$T_{R,PLik}$	D4
4	Acceptance criteria to correlate DLs and PLs	DLi→PLi	D17

5. Seismic hazard

The seismic input can be provided in different forms and the hazard assessment can refer to one or more intensity measures. A proper definition of the seismic demand is addressed by the information and choices that have been assumed in the first step (Classification). In particular:

- Intensity Measure (IM): it depends on the Class of the architectonic asset. Peak Ground Acceleration (PGA) is the most frequently adopted, due to the large amount information (strong motion records) and models (GMPEs) which are available; it is a good parameter in case of buildings (Class A), in particular if they are not too slender and with limited ductility, or other massive structures (Class E). For assets of Classes A, B, C and D the spectral acceleration for a period of vibration meaningful for the asset (elastic or related to DL2) may be a good IM ($S_a(T_{DLk})$, k=1 or 2). If the asset is characterized by a large ductility, integral measures of the seismic input, like for example the Housner Intensity I_H, provide good results, because 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 maximum spectral displacement ($S_d(T_D)$) or the peak ground displacement (PGD). Other IMs, as for instance to the duration of the ground motion, can be considered only if nonlinear dynamic analyses are used for the assessment and in combination with another IM (Vector-Valued PSHA).
- Seismic Input can be described by: 1) an Acceleration-Displacement Response Spectrum (ADRS), completely defined for the specific site of the building under investigation as a function the Intensity Measure (IM); 2) a proper set of time histories, selected from real recorded accelerograms, obtained through numerical models of the seismic source and the propagation to the site or artificially generated, in order to be compatible with a target response spectrum (this last option is quite questionable in case of nonlinear dynamic analysis). The 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, 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 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 Ground Motion Prediction Equations (GMPEs) that represent how the IM attenuates from the epicentre to the site, as a function of their relative distance. As an alternative, the propagation of seismic waves from the fault to the site may be modelled through 3D numerical models, able to consider the fault size and its rupture. By computing a convolution integral of the contributions from all seismic sources, the hazard curve is obtained, a relation between the maximum IM of the seismic input and the annual rate of exceedance λ (or the earthquake return period $T_R \cong 1/\lambda$). In the case of a Vector-Valued PSHA a hazard surface is obtained (Figure 7).

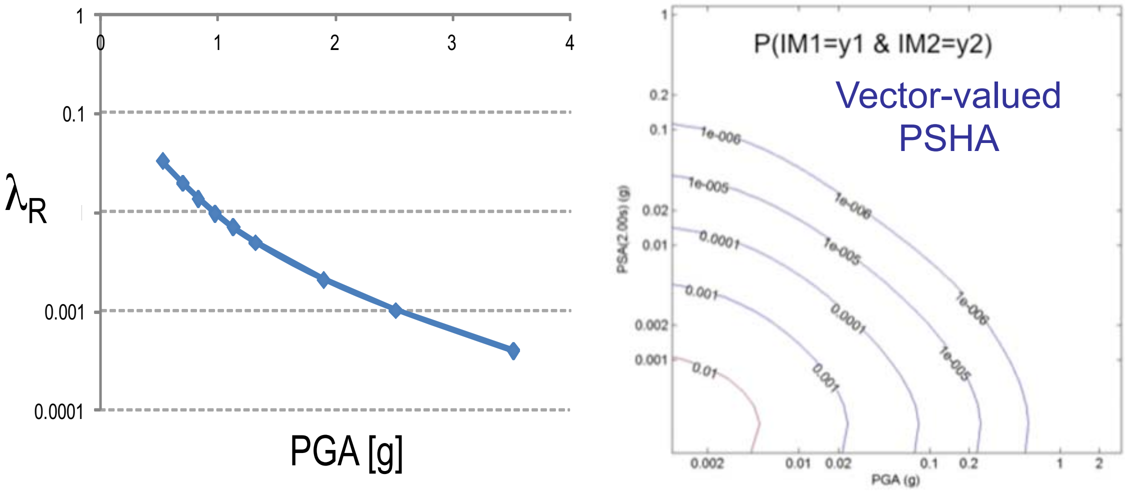

Figure 7. Example of a hazard curve and a hazard surface, from a Vector-Valued PSHA.

The Acceleration-Displacement Response Spectra (ADRS), in which it is assumed the well-know relation $S_d = S_a T^2/4\pi^2$, may be defined:

- analytically, by considering simple formulas in sections between some significant values of the period T;
- through a piecewise linear function, by giving the spectral acceleration S_a values for a given set of period T values.

The latter type is usually obtained when GMPEs are used to attenuate the acceleration response spectrum $S_a(T_k)$ for fixed values of the period T_k (k=1÷N); usually the effects of soil amplification are already included in the GMPEs. In this case, the N parameters $S_a(T_k)$ are IMs and the seismic input is defined through a Vector-Valued PSHA.

With reference to the first type of ADRS, starting from the EC8 format, new amplification factors and soil categories are proposed on the basis of SHARE's global strong-motion database (FP7 project SHARE - Seismic Hazard Harmonization in Europe, 2011). In particular the maximum spectral amplification factor β is introduced (assumed equal to 2.5 in EC8) and a new classification scheme, based on parameters such as average velocity to seismic bedrock, fundamental period and depth of soil deposits is proposed. IM can be, in this case, any of the parameters that define the ADRS; usually the PGA is used. Other parameters are assumed fixed, at least for any earthquake hazard level. EC8 considers two types of response spectra: Type 1, if the earthquakes that contribute most to the seismic hazard defined for the site have a magnitude $M_s \geq 5.5$; Type 2, for low magnitude earthquakes. Usually Type 2 has to be used for PLs related to Damage Level 2.

For architectonic assets that require the adoption of an IM representative of the ADRS in the long periods range (e.g. those of Class F), or more than one IM (Vector-Valued PSHA), the shape of the response spectrum must change, in order to be compatible with the information provided by GMPEs for short a long periods. To this end the following expressions are proposed for the acceleration response spectrum $S_a(T)$:

$$
\begin{array}{ll}
0 \le T < T_B & a_g S \left[1 + \frac{T}{T_B}(\eta\beta - 1)\right] \\
T_B \le T \le T_C & a_g S \eta \beta \\
T_C < T < T_D & a_g S \eta \beta \left[\frac{T_c}{T}\right]^{\alpha} \\
T_D \le T \le T_E & a_g S \eta \beta \frac{T_C^{\alpha} T_D^{2-\alpha}}{T^2} \\
T_E < T < T_F & a_g S \eta \beta \frac{T_C^{\alpha} T_D^{2-\alpha}}{T^2} \frac{T_F - T}{T_F - T_E} + 4\pi^2 \frac{d_g}{T^2} \frac{T - T_E}{T_F - T_E} \\
T \ge T_F & 4\pi^2 \frac{d_g}{T^2}
\end{array} \tag{1}
$$

where: a_g is the peak ground acceleration (PGA); d_g is the peak ground displacement (PGD); S is the soil factor; β is the maximum spectral amplification factor; η is the damping correction factor:

$$
\eta(\xi) = \sqrt{\frac{10}{5 + \xi}} \tag{2}
$$

(with ξ – equivalent viscous damping); α is a proper exponent that joint the response spectrum in the two period ranges (low and long), given by:

$$
\alpha = \frac{log\left(\frac{4\pi^2 S_d(T_D)}{T_D^2 a_g S \eta \beta}\right)}{log\left(\frac{T_C}{T_D}\right)} \tag{3}
$$

where: $S_d(T_D)$ is the maximum spectral displacement. An example of seismic input in terms of expected spectral displacements is the Italian hazard map developed by the INGV-S5 project (Faccioli et al., 2007); the proposed methodology allows to use it in combination with the Italian hazard map developed by INGV in terms of PGA. Reference values for the periods T_B, T_C, T_D, T_E and T_F are proposed in EC8 or in Deliverable D13. Figure 8 shows the typical shape of $S_a(T)$, $S_d(T)$ and $S_a(S_d)$, according to the proposal presented above.

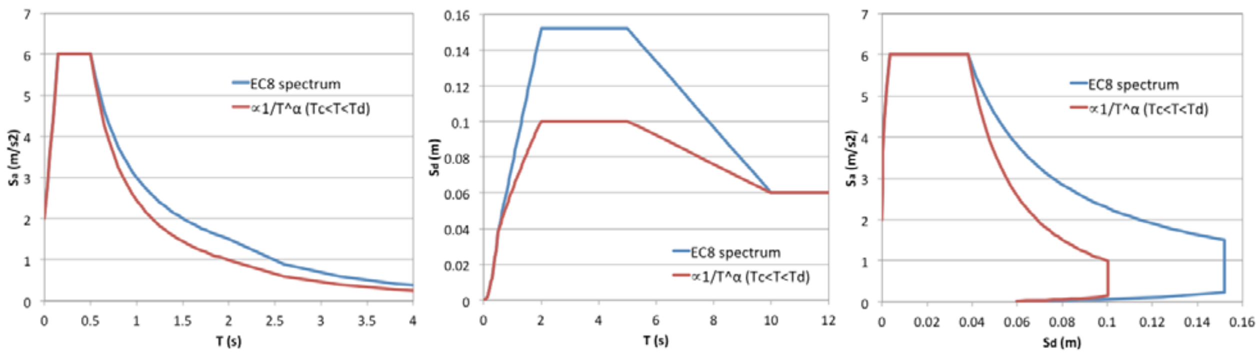

Figure 8. Proposal of analytical Acceleration-Displacement Response Spectra for the PBA.

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, which are reliable also in low frequencies range; the selection is possible from strong motion databases (available on-line) and must refer to parameters as: magnitude, fault mechanism, epicentral distance, soil condition at the site. The first two parameters may be obtained from PSHA, by a disaggregation of the contributions of the different seismic sources, aimed to single out the characteristic of the earthquake that contributes 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.

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 fulfilment of a specific PL (IM_{PLk}), it is necessary to have a sufficient number of cases which produce a seismic demand very close to this condition. In this case a preliminary estimate of the IM is necessary, in order to select useful records, and the minimum number is around 20, for each Damage Level that has to be verified.

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. Also in this case a preliminary estimate of the IM is necessary, as scaling of records is admissible only by using factors that are not too different from one. Differently from the cloud method, with IDA all selected records are used for the evaluation of IM_{PLk}, thus the minimum number of time histories is relatively low (e.g. 7).

Besides being the generation source of earthquakes, tectonic faults may also directly affect surface structures by means of permanent ground displacements. Given their significantly longer lifetime expectancy, their immovability, and in many cases their relatively large size, monuments are more likely than most other structures to experience such tectonic hazards. If the architectonic asset is very close to an active fault, the response to tectonically induced ground distress has to be considered. The differential settlements at the foundation level (values and distribution) can be evaluated by simulating the soil rupturing experimentally or through a detailed nonlinear 3D finite element model; then, the obtained values should be applied at the base of the nonlinear model of the building. The most accurate simulation should consider a global model, able to take into consideration fault rupture–soil–foundation–structure interaction, eventually conducted in two steps through sub-structuring.

The results achieved in this third step are reported in Table 7.

Table 7. Results of step 3: seismic hazard assessment.

Outcome		Parameters	Deliverables
1	Selection of the Intensity Measure IM (one or more than one – Vector-Valued PSHA)	PGA / $S_a(T_2)$ $S_d(T_D)$ / PGD …	D10 D24
2	Hazard curve or hazard surface	$\lambda(IM)$ / $T_R(IM)$	D10
3	In case of nonlinear static analysis (pushover) - parameters for the definition of ADRS	a_g,S,β,α,d_g T_B,T_C,T_D,T_E,T_F	D13 D18
4	In case of nonlinear dynamic analysis (IDA, cloud) - set of time histories (compatible with hazard)	N records (for each DL)	D10
5	In case of potential tectonic faults distress: - distribution of permanent ground displacements from fault rupture–soil–foundation–structure interaction	settlements	D21

6. As-built information

In this sub-step, geometrical, technological and mechanical features of the asset are analysed in depth, with the aim of defining the structural model of the building and related artistic assets.

The as-built information process concerns the acquisition of several data related to: geometry of the building; foundations; mechanical parameters estimate; historical data on transformation and damage (with particular reference to past earthquakes); state of maintenance and damage mechanisms identification (in case of post-earthquake assessment); dynamical behaviour. In PERPETUATE procedure, tools to optimize the investigation program are proposed.

Deliverable D22 presents a detailed state-of-art of recommendations provided by international documents and codes on how to plan investigations (survey, inspections, testing) and take into account of the acquired level of knowledge in the assessment. In case of ancient masonry buildings it is necessary to consider that: 1) in the seismic safety assessment of an existing building epistemic uncertainties due to the incomplete knowledge of the asset, which influence the reliability of models, add up to statistical ones, in particular related to the estimate of material parameters; 2) in cultural heritage assets, the primary conservation objective should be guaranteed and thus the impact of investigations should be minimized; 3) for the development of models, the integrated use of historical data and visual inspections is less invasive and sometimes even more important than the direct quantitative measurements of mechanical material parameters.

Assumed that a proper level of knowledge is always necessary, the idea is that investigation program should be based on preliminary sensitivity analyses aimed to: 1) identify the main parameters to be investigated; 2) define proper Confidence Factors (CFs) of the model, to be used for the assessment in order to take into account uncertainties.

The identification of main parameters influencing the structural response of the asset allows to finalize the investigation to few important points (thus reducing costs and time) and to reduce the number of destructive tests.

The calibration of CFs on the basis of sensitivity analyses instead of a-priori assumptions (as usually done for standard buildings) provides more reliable models and results. In fact, in sensitivity analyses both statistical uncertainties, treated by proper random variables, and epistemic uncertainties, treated by a logic tree approach, are considered.

The following steps summarize the procedure proposed for the sensitivity analysis:

1. Achievement of a “basic” knowledge level of the building, sufficient to identify the most suitable model to be adopted for the seismic assessment.
2. Identification of variables, or groups of correlated variables, which affect the structural response. Variables are related to geometry, mechanical parameters and constructive details (e.g. presence and effectiveness of tie rods).
3. For each variable x_k (k=1..N, where N is the total number of groups of variables related to geometrical data or mechanical parameters), identification of a rational range of variation (that is $x_{k,inf}$ and $x_{k,sup}$). Once specified the range of variation it is possible to define:

$$\bar{x}_k = \frac{x_{k,\inf} + x_{k,\sup}}{2}$$

$$f_k = \frac{x_{k,\sup} - x_{k,\inf}}{x_{k,\inf} + x_{k,\sup}} \qquad (4)$$

   where $\bar{x}_k$ is the mean value and f_k will be used to calibrate the confidence factor on the basis of the actual variation expected for each variable.
4. Some variables are not defined by a range of variation but lead to the adoption of different models (epistemic variables); they are enumerated in the following as Y_i (i=1..M, with M total number of variables leading to the adoption of different models). For each Y_i factor, two possible models can be adopted, quoted as A and B. Thus, in general, M^2 models have to be considered (combination of all possible configurations). In the following, models will be enumerated through a “j” counter (j=1..M^2).
5. Execution of the sensitivity analysis with selected variables, in order to evaluate how much each one really affects the seismic behaviour of the examined building.
6. Attribution to each variable of a “sensitivity class” (low, medium or high), on the basis of the post-processing of results provided from the sensitivity analysis.
7. Plan of the investigations and testing by using the results obtained from steps 5) and 6). The procedure encourages to enhance the investigations on the variables that result in high (or even medium) sensitivity class.
8. Execution of investigations and tests.
9. Definition of the Confidence Factor, possible updating of the mean value of variables (on the basis of tests/investigations results) and final definition of variables to be used in models for the seismic assessment. Confidence Factor is given by:

$$CF = \begin{cases} 1+\beta_{\max} f_c & \text{if the worst evaluation of } IM_{PL} \text{ is for the higher value of } x_c \\ 1-\beta_{\max} f_c & \text{if the worst evaluation of } IM_{PL} \text{ is for the lower value of } x_c \end{cases} \quad (5)$$

where: x_c is one of the variables in high sensitivity class, selected for the application of CF in the PBA; f_c is the parameter defined above; β_{max} is the maximum of sensitivity coefficients evaluated for each variable of the basis of their sensitivity class and the achieved level of knowledge, according to Table 8.

Table 8. Definition of sensitivity coefficient β_{Xk}.

	Sensitivity Class		
Knowledge Level	Low	Medium	High
KL1	0.3	0.6	1
KL2	0	0.3	0.6
KL3	0	0	0

The procedure is described in details in D22, together with an example of application. The example considers 3 random variables and 2 epistemic variables. Table 9 summarizes the results of sensitivity analyses. Figure 9 shows the logic tree of epistemic variables.

Table 9. Summary of attributions of sensitivity classes and knowledge level to each variable.

Variable Type	Set of variables	Variable	f_k	+/-	α		Sensitivity class	Knowledge Level	β_{Xk}
X_k	1	x_{1a}	0.25	Inf	$\alpha_{X,k}$	0.1313	H	KL2	0.6
		x_{1b}	0.25						
	2	x_{2a}	0.25	Inf		0.07221	M	KL2	0.3
		x_{2b}	0.25						
		x_{2c}	0.20						
		x_{2d}	0.20						
	3	x_3	0.20	Sup		0.0037	L	KL1	0
Y_i	Y_1		-	-	α_{Yi}	0.125	H	KL3	-
	Y_2		-	-		0.147	H	KL2	-

Once the plan of investigations and testing is defined, the best techniques must be adopted, taking into consideration both their reliability and the need of minimizing the invasiveness. To this end, a detailed and up-dated review of techniques for the knowledge of cultural assets is presented in D6, with information related to principles and scopes of testing, figures of test set-up, test procedures and measurements, interpretation of results. Moreover, the applicability and the reliability of each technique (graduated on three levels: low, medium, high) are discussed in relation with structural modelling approaches and required mechanical parameters.

One of the main problems in numerical modelling 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 measurement. 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 types of stone and brick masonries, based on available data from literature and new experimental tests (D15).

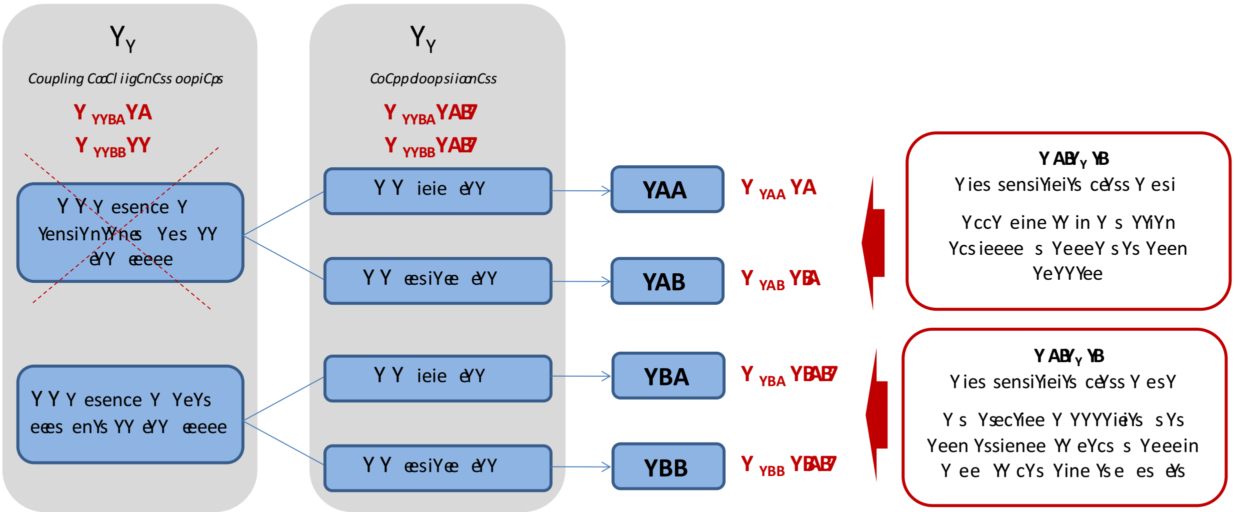

Figure 9. Subjective probabilities assigned to the logic tree branches after sensitivity analysis and investigations carried on epistemic variables.

In order to improve the knowledge of the architectonic asset, with the aim of a more reliable modelling and assessment, ambient vibration tests can be very useful for identifying the overall dynamic behaviour of heritage buildings (structural identification).

Finally, in case of complex and particular assets (e.g. when it is difficult to assess the behaviour of specific constructive details only by a theoretical approach), 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 (D12, D33).

The results achieved in this fourth step are reported in Table 10.

Table 10. Results of step 4: as-built information and sensitivity analysis.

Outcome		Parameters	Deliverables
1	Historical data, transformations, pre-existing damage, distresses	damage modes, settlements	D5 D6 D11
2	Selection of random and epistemic variables. Sensitivity analysis.	x_k (k=1..N) Y_j (j=1..M^2)	D22
3	Plan of investigations and definition of geometrical, constructive and material parameters	Values of model parameters	D6 D15
4	Evaluation of the Confidence Factor and the related variable to which CF has to be applied	CF / x_c	D22
5	Ambient vibration test for structural identification	natural periods, modes, damping	D16
6	Shaking table tests on full-scale or scaled mockups	assessment information	D12 D33

7. Structural models for the seismic analysis and assessment procedures

The outcome of the PBA proposed in PERPETUATE is the maximum seismic Intensity Measure compatible with the fulfilment of each performance level (IM_{PLi}), which are identified in the second step (safety and conservation requirements). 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.
- *Incremental Dynamic Analysis* (IDA) or *Nonlinear Dynamic Analyses* with a large amount of records (cloud method), based on the statistical evaluation of IM_{PLi} from the results of nonlinear dynamic analyses with properly selected time histories.

The first method (CSM) is assumed as the standard one. It can used for all classes of architectonic 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 Deliverable D26.

The second one (by IDA or cloud method), 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 unfeasible (e.g. complex assets in Class B). In these cases, instead of referring to the use of a behaviour factor (q), it is possible to define a simplified capacity curve, by assuming: equivalent periods for the initial (T_1) and light cracked (T_2) conditions, an overstrength ratio and displacement capacities (ductility) for DL3 and DL4. The assessment is then made by the CSM.

7.1 Nonlinear static analysis and Capacity Spectrum Method

With reference to the global behaviour, the procedure aimed to evaluate $IM_{PLi,g}$ may be summarized by 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 in capacity curve; 4) given the seismic demand (in terms of proper IM), computation of the maximum IM value compatible with the *i*-th PL ($IM_{PLi,g}$).

1) Pushover analysis

The pushover analysis can be made, in most cases, by the modelling 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 and for the analysis of macroelements characterized by loss of equilibrium of the structure, considered as a kinematism of rigid blocks, the MBM may be used; a nonlinear kinematic analysis is performed, by incrementing the mechanism displacements of each block and applying a pattern of horizontal loads proportional to the seismic masses; taking into account geometry nonlinearities, a pushover curve is obtained.

In order to practically model the different classes of architectonic and artistic assets, many critical observations and operative tools are provided by Deliverables D26 and D23.

Modelling strategies for masonry foundations and soil-structure interaction are also described (D25). Historical masonry buildings are massive structures and often the foundations are not so deep; the dynamic response certainly interacts with that of the surrounding soil and thus, conventional foundation models based on the non-deformable rigid body hypothesis may be not accurate enough. For slender building typologies, such as towers, the Soil-Foundation-Structure Interaction (SFSI) may produce significant rocking effects and associated damping on the system. For massive high frequency structures the importance of such interactions effects may be equally important. In general, for heavy stiff structures resting on soft soil, linear and nonlinear SFSIs plays an important role on the response of the foundation, transferring stress fields from the structure to the foundation, filtering high frequencies and hence modifying the response of the building, both static and dynamic. As a matter of fact, historical masonry buildings have a foundation system that can transfer negligible tensile stress and a limited bending moment. The actual flexibility and geometry of the foundation system, in combination with the nonlinear behaviour of the foundation soil, may be considered through proper impedance functions (developed by PERPETUATE on basis of detailed numerical analyses), which can be implemented in the model (in particular SEM).

The structural modelling of the architectonic asset can lead to the definition of a single model (simple assets, made by a single macroelement, or complex assets, characterized by a box-behaviour) or many models (complex assets made by almost independent macroelements). For simple assets, the single macroelement is modelled and the assessment is performed on its pushover curve. For complex assets, it is necessary to distinguish the following two sub-cases:

- Buildings characterized by box behaviour. In this case, a 3D model of the whole building is required (*global scale* approach) and the assessment is performed on its overall capacity curve.
- Buildings made by a set of N macroelements, which exhibit an almost independent behaviour. In this case, each macroelement is modelled independently (*macroelement scale* approach) and N capacity curves are evaluated, the seismic load being assigned by a proper redistribution. The assessment of whole asset is then made through proper combination criteria (see §7.3).

Figure 4 summarizes the relationship between the two types of assets (simple and complex) and the related seismic assessment scales (macroelement and global).

2) Positioning on the pushover curve of Damage Levels and related Performance Levels

In order to check the fulfilment of Performance Levels (PLs), Damage Levels (DLs) have to be positioned on the pushover curve, by considering all information provided by the incremental nonlinear static analyses. This a complex task, which is faced in codes and recommendation documents according to the following main approaches:

- *Structural element approach*. It assumes that the attainment of a certain DL_i at global scale corresponds to the step in which the first structural element reaches the same damage level.
- *Heuristic approach*. DLs are directly defined on the capacity curve on the basis of conventional limits, usually expressed in terms of interstorey drift and decay percentage of the overall base shear (in respect to its maximum value).

In case of cultural heritage buildings, the application of these approaches may lead to unreliable results. Indeed, while in case of a box-type behaviour with quite rigid horizontal diaphragms it is reasonable that many elements and walls reach almost at the same time a certain DL_i, in case of existing buildings this condition is far to be true, due to the architectural complexity and the presence of flexible floors. In the limit case of completely flexible floors, it is reasonable assuming a quite independent behaviour of each single masonry wall (or macroelement), and the reaching of a certain DL_i must refer to the macroelement scale rather than the global one. However, this is a limit condition and in general, even in case of wooden floors or vaults (that are typical for monumental buildings), horizontal diaphragms provide a not negligible contribution to the load transfer and stiffness. As a consequence, the reaching of a certain DL_i at scale of a single wall may not appear evident in the pushover curve of the whole structure. The lack of homogeneity of damage in case of very complex buildings is even more evident at the scale of masonry elements (piers and spandrels); thus the reaching of a severe DL in a single element could not correspond to a significant strength decay on the overall capacity curve.

The effects of adopting the classical approaches to the definition of DLs are double: a) the *structural element approach* could lead to too cautionary assessment; b) the *heuristic approach* may result quite conventional and not cautionary if adopted as single criterion to define the DLs on the capacity curve (in particular, it does not assure from the occurrence of heavy DLs at local or macroelement scale). All the above mentioned issues stress the need to outline a procedure able to take into account all the different scales that concur to define the overall seismic response and to combine different criteria.

The multicriteria approach proposed in PERPETUATE (D17) aims to take into account the response of asset at different scales: structural elements scale (local damage), architectonic elements scale (damage in macroelements) and global scale (pushover curve). The basic idea is the assessment of a certain DL_i at global scale comes out from a set of checks carried out at different scales, based on: a heuristic approach related to the attainment of conventional limits on the capacity curve (fixed by expert judgment or parametric analyses), either of the whole building or of “relevant” macroelements; systematic checks on elements and macroelements, aimed to avoid the occurrence of local damage levels incompatible with the fulfilment of the considered DLi.

Let be considered, as an example, the assets of class A (*Assets subjected to prevailing in-plane damage*). These buildings are usually complex assets made by many macroelements and are characterized by a box behaviour. Thus, their seismic assessment is performed at the scale of the whole building (global scale). The checks should be related to a set of variables that describe, in correspondence with the current displacement *d* on the pushover curve: 1) the state of the whole building; 2) that of macroelements (e.g. each masonry wall); 3) the damage in structural elements (piers or spandrels). The variables chosen in PERPETUATE are: 1) the percentage of overall base shear decay β_G at global scale; 2) the interstorey drift δ_I or the percentage of base shear decay β_M, at macroelement scale; 3) the drift δ_E, or the percentage of strength reduction β_E, in single elements or the cumulative rate of elements (piers - $\beta_{cum,P}$ - and spandrels - $\beta_{cum,S}$) that reach a certain DL_i, at element scale. Table 11 collect the proposed conditions that have to be checked at the different scales. Figure 10 illustrates synthetically the path to be followed.

Table 11. Definition of DLs by multicriteria approach, by checking at the different scales some limit thresholds (all limits are in %).

DLi	Structural element scale		Macroelement scale	Global scale
	Piers	Spandrels		
1	-	-	$\delta_{is,1} \in (0.05,0.15)$	-
2	maximum strength $\beta_{cumP,2} \in (2,3)$	-	Max base shear (V_M) $\delta_{is,2} \in (0.15,0.3)$	-
3	$\beta_{se,3} \in (0,20)$ $\delta_{se,3} \in (0.3,0.6)$ $\beta_{cumP,3} \in (2,3)$	$\beta_{se,3} \in (0,50)$ $\delta_{se,3} \in (0.5,1)$ $\beta_{cumS,3} \in (3,4)$	$\beta_{M,3} \in (30,40)$	$\beta_{G,3} \in (10,20)$
4	$\beta_{se,4} \in (20,60)$ $\delta_{se,4} \in (0.4,1.2)$ $\beta_{cumP,4} \in (2,3)$	$\beta_{se,4} \in (20,70)$ $\delta_{se,4} \in (1,2)$ $\beta_{cumS,4} \in (3,4)$	$\beta_{M,4} \in (60,80)$	$\beta_{G,4} \in (30,40)$
5	$\delta_{se,5} \in (0.6,1.5)$ $\beta_{cumP,5} \in (2,3)$	$\delta_{se,5} \in (1.5,2.5)$ $\beta_{cumS,5} \in (3,4)$	-	-

As regard the structural elements, drift and strength decay limits have to be differentiated as a function of the different failure modes that may occur (e.g. Rocking, Diagonal Cracking, Bed Joint Sliding or mixed modes as well); moreover, in the case of spandrels, as highlighted by some recent experimental campaigns (Beyer, 2012), these values are also function of the architrave type (e.g. masonry arch, stone lintel or wooden/steel beam) and of the presence of another tensile resistant element coupled to it (e.g. tie-rod or r.c. ring-beam). Proper multi-linear constitutive laws must be used, in order to get a reliable pushover curve till to near collapse condition. Figure 11shows two examples: a) a pier with a prevailing shear response (e.g. Diagonal Cracking); b) a spandrels with wooden lintel (Cattari and Lagomarsino, 2012).

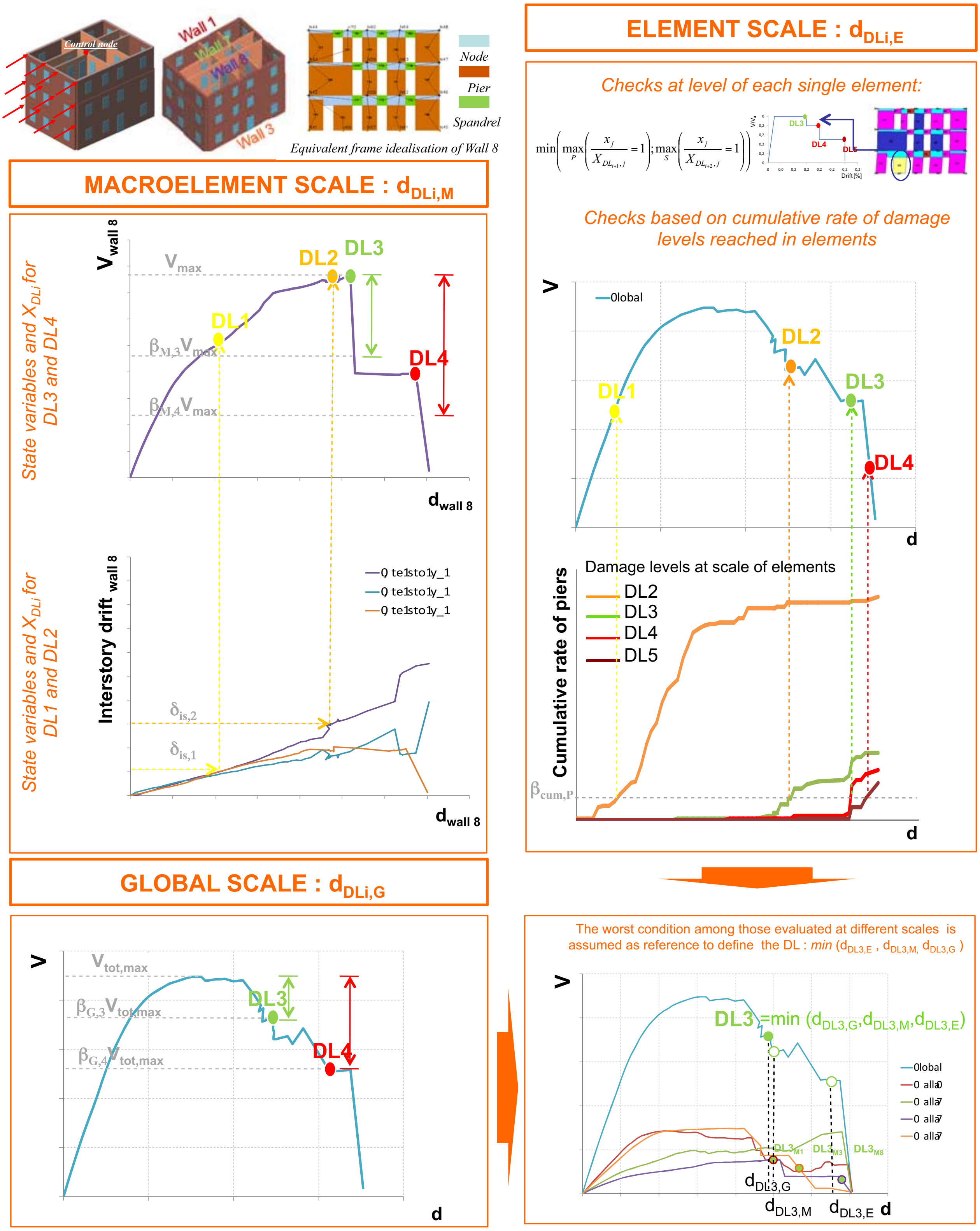

Figure 10. Multi-criteria approach to DL identification in the case of buildings of class A.

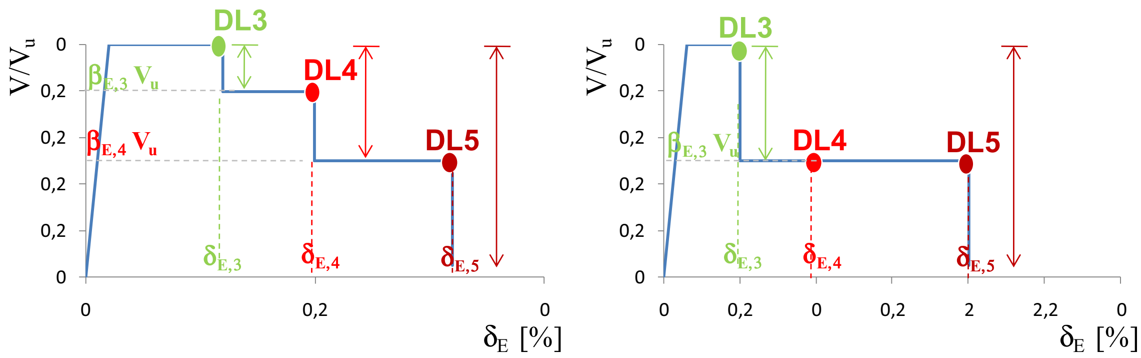

Figure 11. Example of multi-linear constitutive laws for piers (left) and spandrels (right) elements, with the definition of damage levels.

Proper criteria to define DLs on the pushover curve obtained by Macro Block Models (MBM) are also proposed. In this case, the conventional limits proposed in the framework of the heuristic approach have been validated through many nonlinear dynamical analyses performed on Housner-like rigid blocks models or degrading nonlinear elastic systems. In addition, some criteria at element scale can be added, referred to local critical conditions that can occur with the progression of the mechanism. Table 12 and Figure 12 show the proposed criteria and two typical examples of pushover curve obtained by nonlinear kinematic analyses.

Table 12. Definition of DLs in case of nonlinear kinematic analysis by MBM.

DLi	Single block or Single Macroelement
1	In terms of percentage of the horizontal multiplier associated to d_{DL2} d_{DL1} corresponds to the point in which the multiplier is $\alpha_{DL1}=0.7\alpha_{DL2}$
2	In terms of percentage of d_y and check on d_{peak} $d_{DL2} = \min(\beta d_y\,;\, d_{peak})$ - with $\beta \in (2,4)$
3	In terms of percentage of the ultimate displacement capacity d_0 $d_{DL3}= 0.25\, d_0 \geq d_{DL2}$
4	In terms of percentage of the ultimate displacement capacity d_0 $d_{DL4}= 0.4\, d_0 \geq d_{DL2}$

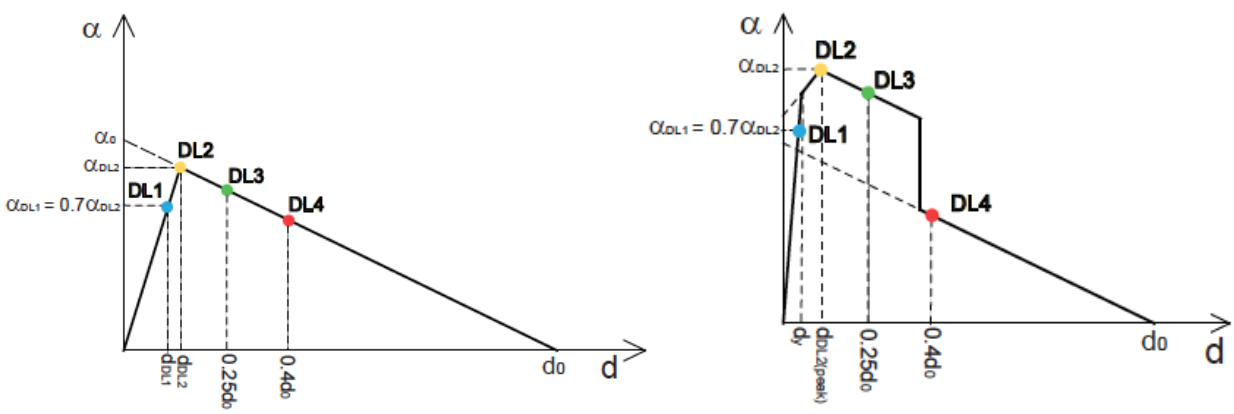

Figure 12. Criteria to define DLs on the pushover curve in case of nonlinear kinematic analysis by MBM.

Once defined DLs on the pushover curve, they have to be properly correlated to the corresponding PLs according to criteria introduced in §4 (see Figure 6).

3) Conversion of the pushover curve into a capacity curve

The pushover curve obtained from the multi degrees of freedom model of the asset is converted into the capacity curve of the equivalent single degree of freedom system (SDOF), which can be compared with the seismic demand.

In the case of nonlinear static analysis, the Γ coefficient is used, as proposed by Fajfar in the N2 Method (Fajfar, 2000) and adopted in Eurocode 8 (2004), which is defined by the following expression on the basis of a deformed shape ϕ, assumed as representative of the collapse mode:

$$\Gamma = \frac{\sum m_i\ \phi_i}{\sum m_i\ \phi_i^2} = \frac{m^*}{\sum m_i\ \phi_i^2} \tag{6}$$

where m_i is the mass of the i-th storey/block, m^* is the equivalent mass of the SDOF system and ϕ_i is the component of the deformed shape ϕ (in which the components are normalized in way that ϕ_k=1, where k is the displacement of the control node). In the case of class C (usually dominated by the first mode), it seems appropriate to assume ϕ equal to the fundamental modal shape; it turns out that Γ coincides with the participation factor of the first mode mass.

The capacity curve is defined in terms of spectral acceleration (a*) and spectral displacement (d*) of the SDOF, computed as follows:

$$a^* = \frac{V}{\Gamma m^*}$$
$$d^* = \frac{d}{\Gamma} \tag{7}$$

where V is the total base shear and d the displacement of the control node.

In the case of nonlinear kinematic analysis, being already the mechanism a single degree of freedom system, the block displacements are assumed as deformed shape for converting the original pushover curve, defined by the horizontal force multiplier α, into a capacity curve:

$$a^* = \frac{\alpha g}{\Gamma m^*} \tag{8}$$

where g the acceleration gravity.

Figure 13 summarizes the conversion of the pushover curve into the equivalent capacity curve for the two above-mentioned cases.

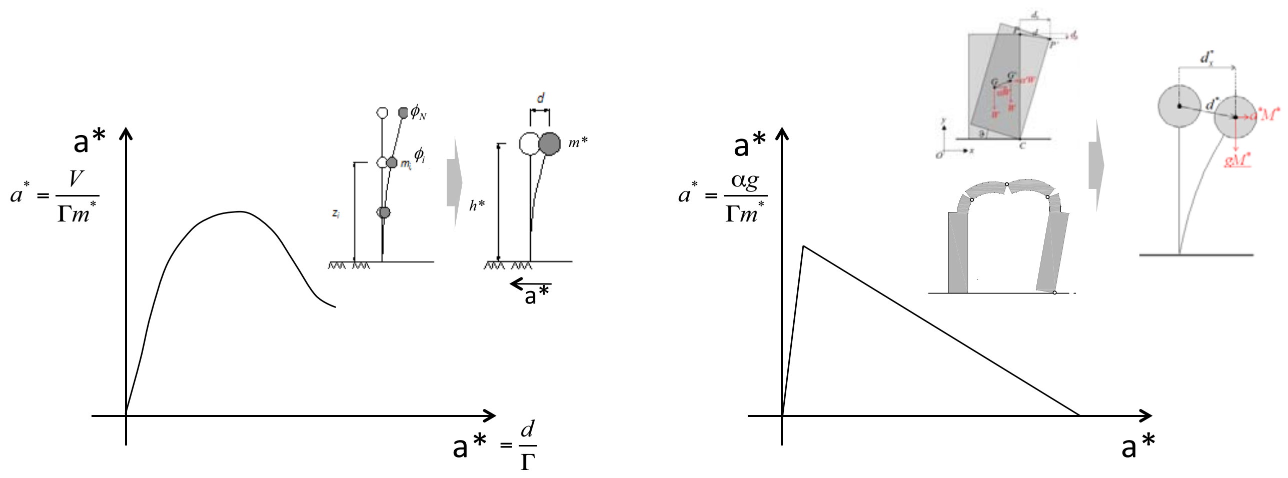

Figure 13. Capacity curves in case of both nonlinear static and kinematic analyses.

4) Evaluation of $IM_{PLi,g}$ - maximum IM compatible with the i-th PL

Different methods are available in the general framework of the Capacity Spectrum Method, that is the evaluation of the displacement demand on the capacity curve given an Acceleration-Displacement Response Spectrum (CSM: Freeman et al., 1975; Freeman, 1998; N2 Method: Fajfar, 1999 and 2000; Coefficient Method: ASCE/SEI 41-06; MADRS Method: FEMA 440; Displacement-based method: Calvi, 1999).

The method proposed by 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, coherent with the hysteretic dissipation due to nonlinear behaviour). If the seismic input is given, the evaluation of the displacement demand requires an iterative procedure.

PERPETUATE procedure is simpler and direct, as it requires the evaluation of the maximum IM compatible with the *i*-th given PLi ($IM_{PLi,g}$). To this end, it is sufficient:

1. to define the period (T_{PLi}) and damping coefficient (ξ_{PLi}) associated to each PLi, as defined on the capacity curve;
2. to define proper criteria to compute $IM_{PLi,g}$.

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

$$T_{PLi} = 2\pi\sqrt{\frac{a^*_{PLi}}{d^*_{PLi}}} \qquad (9)$$

The values of damping coefficients ξ_{PLi} may be estimated by different approaches:

- from cyclic pushover analyses (Figure 14), by evaluating the hysteretic dissipation ξ_{hyster}, and adding a constant viscous damping ξ_0 (usually assumed equal to 5%, except for Class F, which is characterized by lower values – 2%), or on basis of results of nonlinear dynamic analyses;

$$\xi_{hyster} = \frac{1}{4\pi}\frac{E_D}{E_{S0}} \qquad (10)$$

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

$$\xi_{PLi}=\xi_0+\xi_{hyster,max}\left(1-\frac{1}{\mu_i^{\beta}}\right) \qquad (11)$$

  where: $\mu_i = d^*_{PLi}/d^*_{PL1}$ is the ductility; $\xi_{hyster,max}$ is the asymptote of the hysteretic damping and β is a coefficient that modifies the rate of increase of hysteretic damping with ductility (reference values are suggested in Table 13);

- from results of experimental tests on the examined building or on analogous structures, taking into consideration this method could provide a significant underestimation.

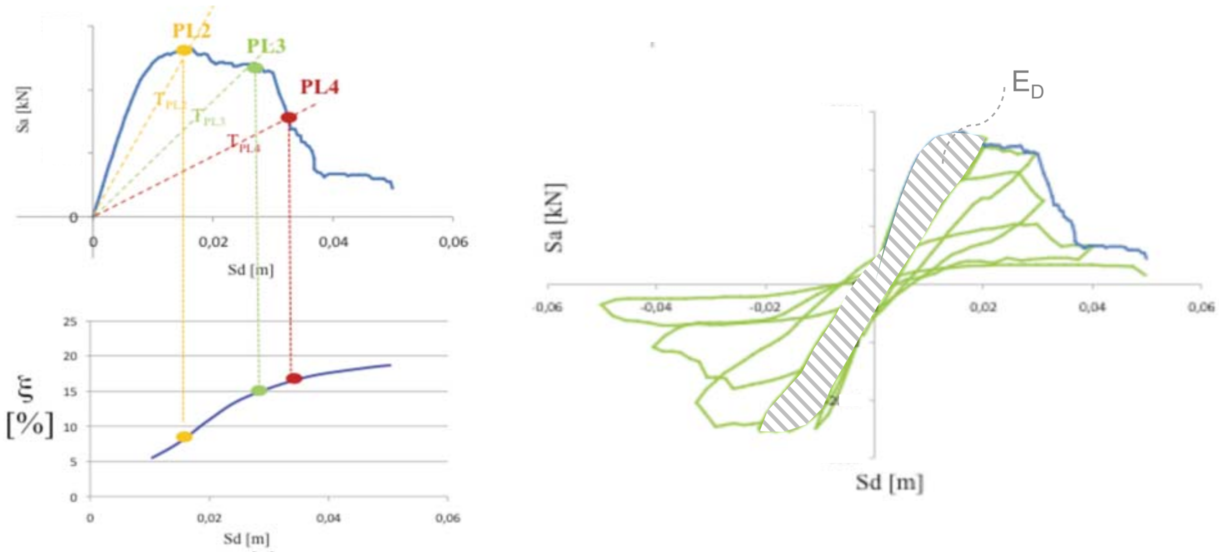

Figure 14. Evaluation of equivalent viscous damping ξ_{PLi} associated to different PLi by cyclic pushover.

Table 13. Reference values for $\xi_{hyster,max}$ and β for different Classes of architectonic assets.

	Architectonic Asset Class				
	A	B	C	D	F
$\xi_{hyster,max}$ (%)	25	20	15	15	5
β	1.5	2	1.5	1.2	1

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

$$S_d(T,IM,\xi)=IM\cdot S_{d0}(T,\xi) \qquad (12)$$

If the ADRS is regular and monotone with the period T, $IM_{PLi,g}$ can be simply evaluated as the IM for which the spectral displacement demand $S_d(T_{PLi},IM,\xi_{PLi})$ is equal to d^*_{PLi} (Figure 15a). In order to extend the CSM to the application in the case of irregular ADRS (Figure 15b), the following expression in proposed for the evaluation of $IM_{PLi,g}$:

$$IM_{PLi,g} = \frac{d^*_{PLi}}{\max\left[S_{d0}(T,\xi_{PLi});T_{PL1} < T < T_{PLi}\right]} \tag{13}$$

ADRS similar to the one of Figure 15b could result from the seismic hazard analysis if site is subjected to soil amplification phenomena, which are numerically modelled, or in the case of verification of local mechanisms or artistic assets, located at un upper level of the building and thus subjected to a filtered seismic input (floor spectrum), different from the ground motion one. On the contrary, it is worth noting that this method for the PBA (nonlinear static analysis and CSM) should not be used with ADRS directly obtained from real time histories, because of their shape could be very irregular (high fluctuation in the frequency content).

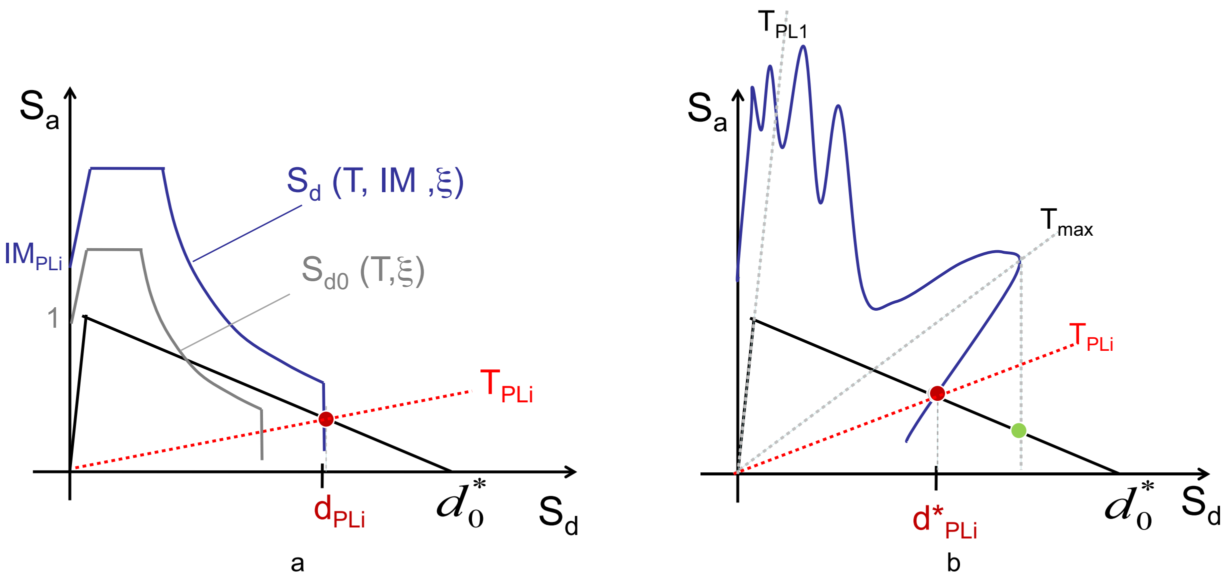

Figure 15. General CSM procedure, proposed for the evaluation of IM_{PLi} with any ADRS.

7.2 Nonlinear Dynamic Analyses (IDA, cloud method)

Nonlinear dynamic analysis is a more accurate method for the seismic assessment, because it reproduces the dynamic behaviour of the structure and does not need the conventional transformation to an equivalent single degree of freedom system (so the contribution of all modes is considered, as well as the effect of vertical component of the input motion, sometimes not negligible). However, the high computational effort and some additional required features (e.g. cyclic hysteretic behaviour of structural elements, not needed for pushover analysis) make the method feasible only for some classes or in specific cases.

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 fulfilled or not. This is not a simple task, as many alternative approaches may be adopted, usually referred to the definition of a global damage index, to be correlated with the PLs. In order to be compatible as much as possible with static nonlinear procedure (pushover-CSM), described in the previous section, PERPETUATE proposes to make reference to a scalar variable:

$$Y_i = \frac{d_{max}}{d_{PLi}} \tag{14}$$

where, according to notation introduced in case of nonlinear static analyses, d_{max} is the maximum value attained during the time history of the representative displacement d (the one used in the pushover curve) and d_{PLi} is the limit threshold that guarantees the fulfilment of PLi. The values of d_{PLi} are defined by the application of the above-mentioned multicriteria approach. Since, the pushover curve depends on the load pattern, results provided by nonlinear dynamic analyses are very useful to validate and, if necessary, modify it.

It is assumed that, after processing the results of a nonlinear dynamic analysis, the attainment of Y_i=1 indicates the reaching of the PLi. The results of a series of nonlinear dynamic analyses may be represented in a IM–Y graph.

If the PBA is made by the Incremental Dynamic Analyses (IDA) procedure, a set of N records is used. The *k-th* record is incrementally scaled through IM until the threshold Y_i=1 is achieved, and the correspondent $IM_{PLi,k}$ (k=1..N) is obtained. IDA curves are plotted in Figure 16. The outcome of the PBA, that is the maximum value of IM that is compatible with the fulfilment of the given PLi is obtained as:

$$IM_{PLi,g} = \frac{1}{N}\sum_{k=1}^{N} IM_{PLi,k} \tag{15}$$

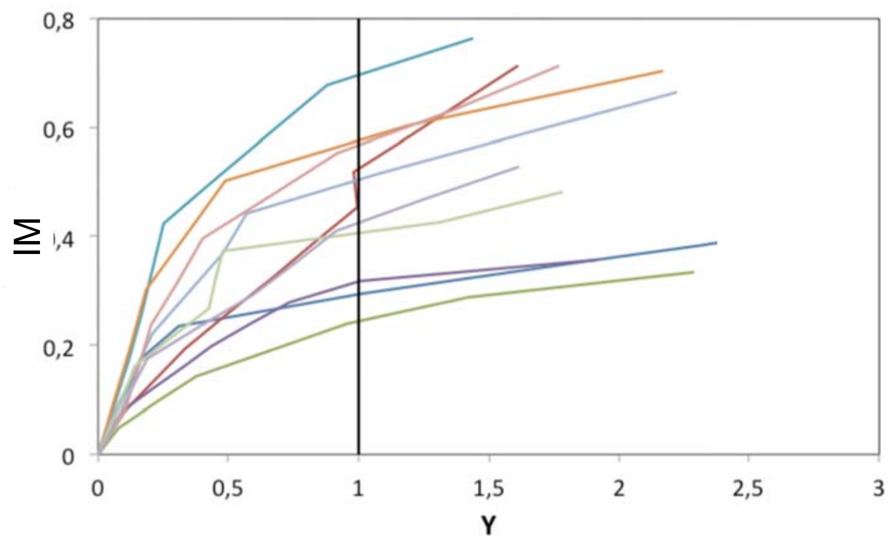

Figure 16.Results of IDA.

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 (the reason is that scaling of records is questionable). In this case it is necessary that many of them cause a response with a value of Y_i close to 1. If a relatively limited range is assumed (e.g. $0.95<Y_i<1.05$), $IM_{PLi,g}$ can be evaluated as the average value of the $IM_{PLi,k}$ (k=1..N) that fall in that range.

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

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

After having computed a proper seismic load sharing, the seismic behaviour of each macroelement is analysed independently and the assessment is performed on the N capacity curves, representative of each n- th macroelement (n=1..N). Proper combination criteria are introduced to define the performance at the scale of the whole asset, if at all excluding some minor macroelements that are considered not relevant for the fulfilment of the examined performance level.

It is worth noting that the subdivision into macroelements must consider to orthogonal directions of the seismic action. Sometimes the same macroelement (proper assembly of elements of the construction) can be considered in both direction, but not necessarily has to analysed by the same model (e.g. the façade of a church can be modelled by CCLM or SEM when loaded in-plane and by MBM for analysing its out-of-plane behaviour).

A 3D linear finite element model of the whole structure is useful to support the reliability of the subdivision in macroelements and to evaluate the redistribution of seismic actions among them (Figure 17).

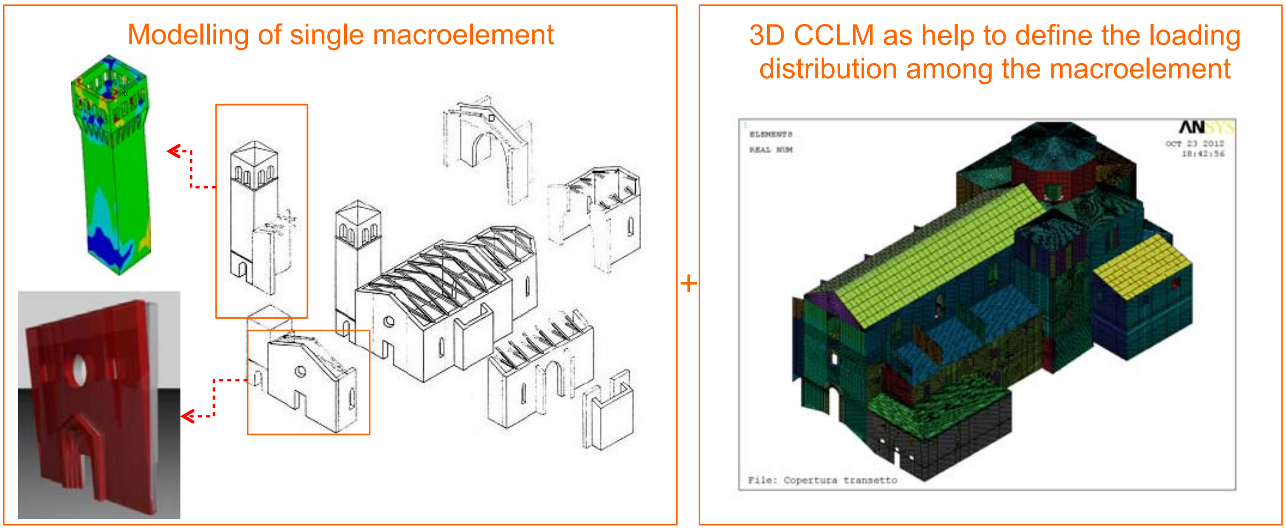

Figure 17. Subdivision into macroelements and redistribution of seismic actions by using a 3D CCLM model.

The PBA of each macroelement is made with the procedure described in §7.1 (evaluation of the pushover curve, positioning of PLs, transformation to a capacity curve). 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. Through the 3D linear model of the asset (considering a seismic load pattern proportional to the masses or multi-modal), or even by simplified assumptions, proper redistribution coefficients α_n have to be evaluated, under the condition that:

$$\sum_{n=1}^{N} \alpha_n = 1 \qquad (16)$$

where N is the number of macroelements.

In particular, stiffer macroelements attract more actions (α_n>1), while the others have to sustain a lower rate of seismic forces (α_n<1). These coefficients are directly used in the PBA, as they modify the capacity curve. Figure 18 shows that if α_n>1 the acceleration capacity a* is reduced (red lines), while macroelements in which α_n<1 take profit of the redistribution (blue lines).

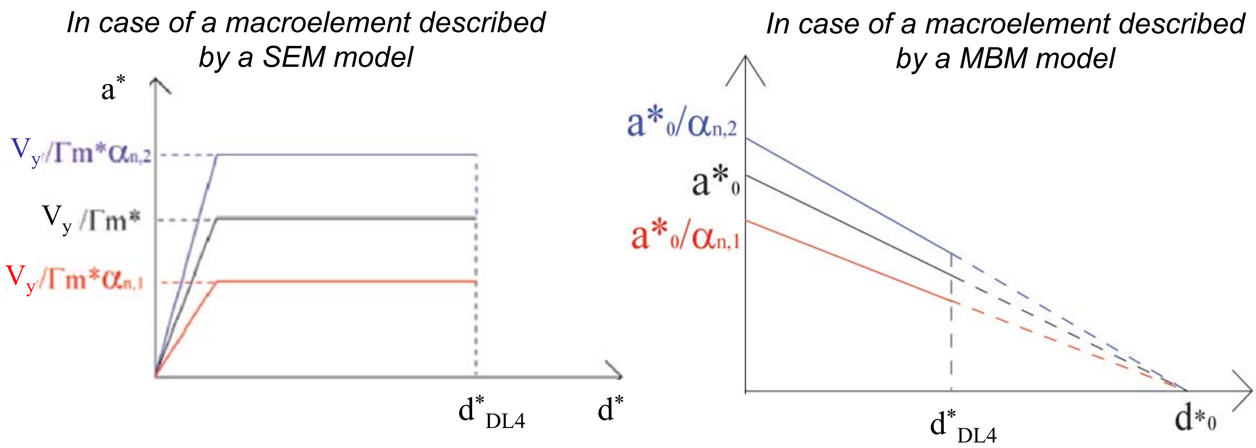

Figure 18 – Capacity curves modified on basis of $\alpha_{1,n}$ and $\alpha_{2,n}$ coefficients

Starting from the seismic assessment carried out on each single macroelement (that is the evaluation of the $IM_{PLi,n}$), it is necessary to introduce proper criteria to assess the maximum demand compatible for the whole asset ($IM_{PLi,g}$).

The simplest method (precautionary) is to assume $IM_{PLi,g}$ as the minimum value provided by the assessments made on the whole set of macroelements, that is: $IM_{PLi,g} = \min(IM_{PLi,n})$ (n=1..N).

In order to be consistent with the multicriteria 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_n has to be assigned to each macroelement, as a function of its relevance in the building with reference to the examined performance; the sum of ρ_n (n=1..N) is equal to 1. Then a fragility curve representative of the seismic performance of the whole asset is evaluated as:

$$P_{PLi}(IM) = \sum_{n=1}^{N} \rho_n \mathbf{H}(IM - IM_{PLi,n}) \qquad (17)$$

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

Finally, the value of $IM_{PLi,g}$ is obtained as the minimum of the following two conditions:

- the lower value of IM for which the fragility curve has $P_{PLi}(IM)\geq 0.5$;
- the value of IM for which the fragility curve of PLi+1 is greater than 0 ($P_{PLi+1}(IM)>0$).

Figure 19 summarizes the procedure proposed to compute $IM_{PLi,g}$ at the scale of the whole asset: in case a) the second condition prevails, whereas in case b) the first one is crucial.

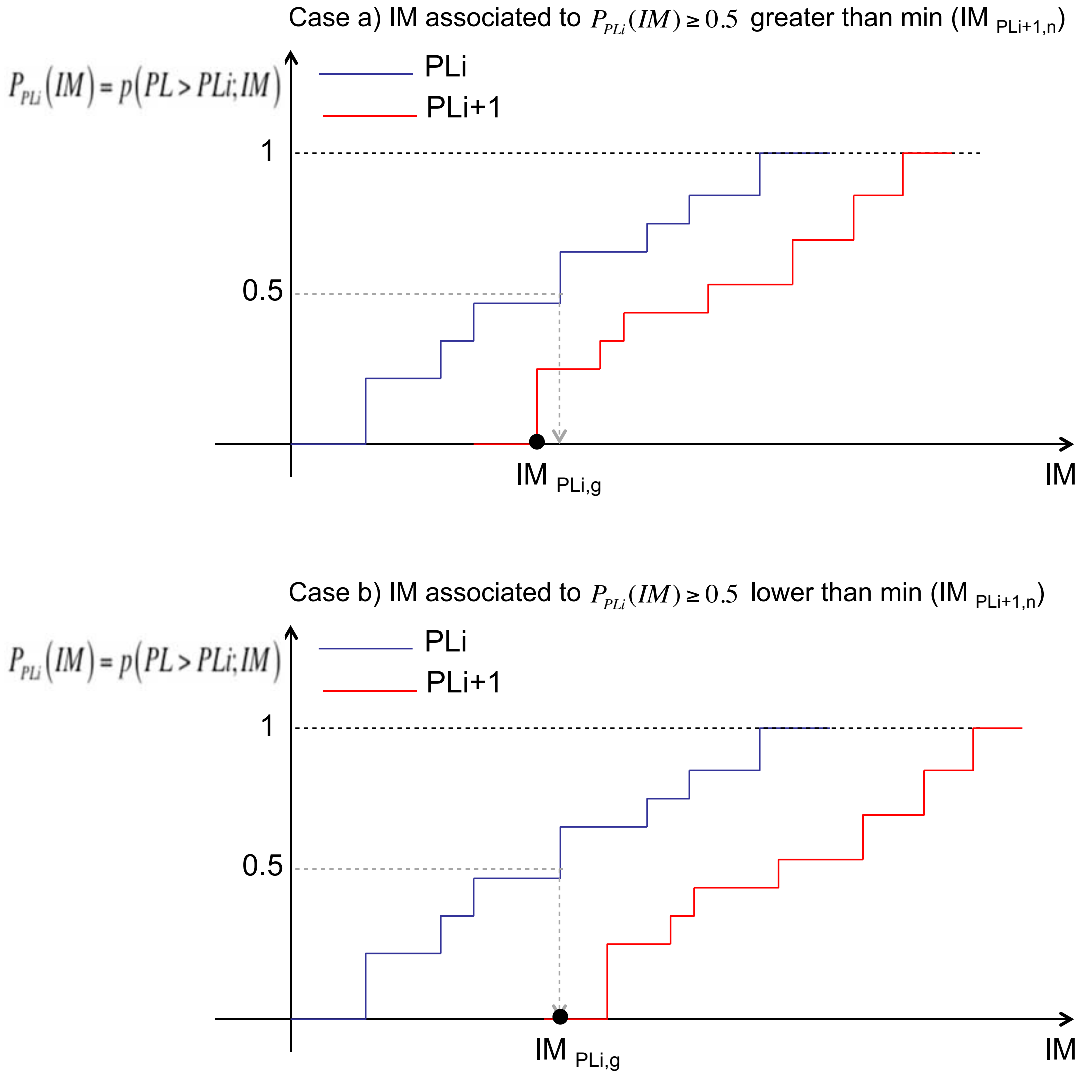

Figure 19. Evaluation of IM_{PLi} in case of assets described by N capacity curves.

7.4 Seismic assessment in case of possible local mechanisms

All the previous issues implicitly refer to the assessment of the building as a whole, the total mass of the structure being involved in the seismic loads. 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” refer to mechanisms that involve only a portion of the building and may not be accurately analysed by the global assessment (for this reason they are studied by a local model, by considering only a fraction of the total mass). Usually, they are treated with the same approach adopted for single macroelement asset. In general, these mechanisms have to be considered only if relevant for the PLs of the examined asset. If M local mechanisms have been selected for PLi, the PBA of these mechanisms supplies, as for that at the global asset scale, values of the maximum IM that is compatible with the fulfilment of PLi: $IM_{PLi,m}$ (m=1..M). The final outcome of the PBA is given by:

$$IM_{PLi} = min[IM_{PLi,g} \quad ; IM_{PLi,m} \ (m = 1..M)] \qquad (18)$$

where $IM_{PLi,g}$ is the value obtained at the global scale.

Since local mechanisms usually interest 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 formulations (*floor spectra*) and after an in-depth calibration supported by dynamic analyses, the following expressions are proposed in PERPETUATE guidelines:

$$S_{dm}(T) = \max\left[\quad S_d(T) \quad ; \quad \sum_{k=1}^{r} S_{dm,k}(T) \quad \right] \tag{19}$$

where: $S_d(T)$ is the displacement response spectrum of the ground motion; r is the number of considered building modes; $S_{dm,k}(T)$ is the contribution of *k-th* building mode to the displacement response spectrum of the filtered acceleration time history at level z, which is given by:

$$S_{dm,k}(T) = \begin{cases} S_d(T_k)\left|\psi_k(z)\right|\gamma_k \dfrac{\left(\dfrac{T}{T_k}\right)^2}{\sqrt{\left(1-\dfrac{T}{T_k}\right)^2 + \dfrac{0.04}{[\eta(\xi)\eta(\xi_m)]^2}\dfrac{T}{T_k}}} & T \le T_k \\ S_d(T_k)\left|\psi_k(z)\right|\gamma_k \eta(\xi)\eta(\xi_m) \dfrac{\left(\dfrac{T}{T_k}\right)^2}{\sqrt{\left(1-\dfrac{T}{T_k}\right)^2 + 0.04\dfrac{T}{T_k}}} & T_k < T < 1.9T_k \\ 3.8\, S_d(T_k)\left|\psi_k(z)\right|\gamma_k \eta(\xi)\eta(\xi_m) & T \ge 1.9T_k \end{cases} \tag{20}$$

where: T_k , ψ_k and γ_k are the *k-th* period, modal shape and coefficient of participation, respectively; $\eta(\xi)$ and $\eta(\xi_m)$ are the damping correction factors related to the dissipation in the building and the mechanism, respectively.

Figure 20 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 the comparison between numerical (continuous lines) and analytical (dashed lines) floor spectra, by varying the fundamental period T of the tower.

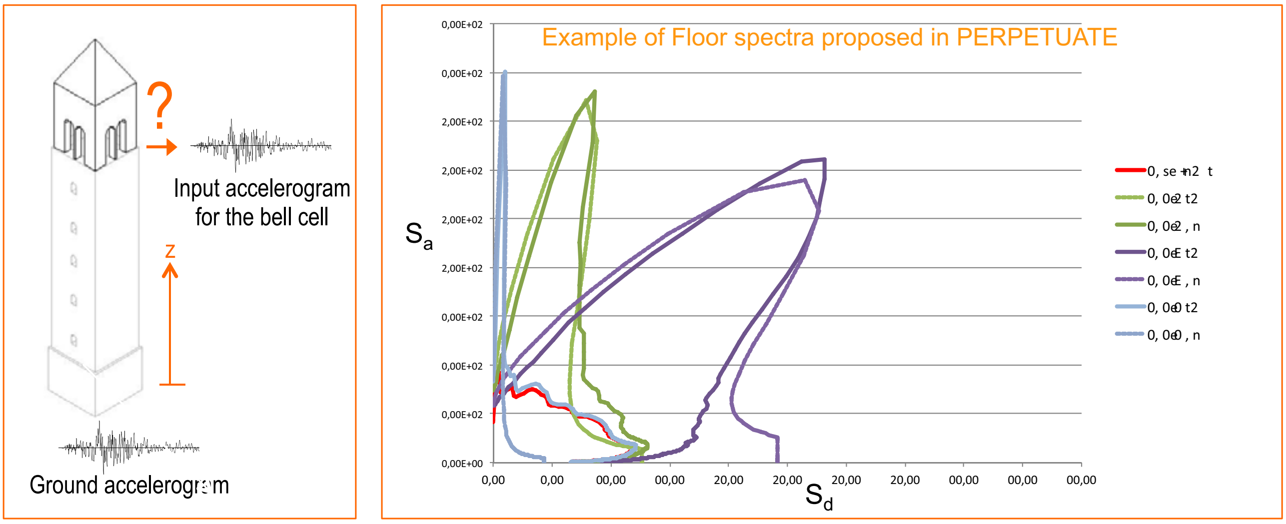

Figure 20.Example of a local mechanism for which it is necessary to evaluate amplified ADRS spectra.

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:

- Class P - artistic assets which are structural elements by themselves (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), in case by adopting specific values related to the consequences to the artistic asset;
- Class Q - artistic assets strictly connected to structural elements (e.g. frescoes, mosaics, stuccoes): their performance is defined by other parameters than the structural ones, but a direct correlation between them can be established (e.g. in case of frescoes, a parameter for measuring the damage level could be the maximum width of cracks, which is correlated to the drift of the panel where the decorated plaster is applied);
- Class R - artistic assets with independent seismic response (e.g. sculptures, balconies, pinnacles, merlons): usually these assets can be modelled by MBM and the PBA must consider amplified ADRS (introduced in the previous paragraph), because of their position in 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 where the artistic asset is placed and, if needed, at the macroelement scale. 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 architectonic asset (Figure 1) or that of the macroelement (in the case of assets described by more than one capacity curve).

7.6 Summary of results

The results achieved in this step are reported in Table 14.

Table 14. Results of step 7: structural modelling and performance based assessment.

Outcome		Parameters	Deliverables
1	Definition of the model	Constitutive laws Impedance functions	D7 D14 D23 D25 D26
2	Global pushover curve	Load pattern $V / \alpha / d$	D35
3	Subdivision of complex assets in macroelements and redistribution of actions for pushover curves	N / α_n	D26 D35
4	Damage Levels and Performance Levels	DL→PL $d_{PLi} / T_{PLi} / \xi_{PLi}$	D17 D35
5	Global capacity curve (or set of N macroelements capacity curves)	$\phi / d^* / a^*$	D26 D29 D35
6	Capacity curves of the M relevant local mechanisms selected	$M / d_m^* / a_m^*$	D26 D29 D35
7	Maximum IM compatible with a given PL	$IM_{PLi,g}$ $IM_{PLi,n} / \rho_n$ $IM_{PLi,m}$ IM_{PLi}	D35
8	Amplified ADRS for the verification of local mechanisms and artistic assets	$S_{d,m}(T)$ at level z r modes	D36

8. Rehabilitation decisions

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

A safety index $I_S = T_{R,PLi}/T^*_{R,PLi}$ can be defined, being greater than 1 when the safety requirements are fully satisfied. It allows defining 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, defined as the number of years during which the building is able to fulfil the assumed performance levels, as long as regular maintenance is provided (Recommendations P.C.M. 9/2/2011, 2011). It is given directly by:

$$V_N = 50\frac{T_{R,PLi}}{T^*_{R,PLi}} = 50\, I_S \qquad (21)$$

Since hazard levels are usually defined for probabilities of exceedance in 50 years, it may be assumed that, if V_N > 50 years, the seismic performance of the architectonic asset is adequate, while, if V_N < 50 years, rehabilitation decisions have to be taken.

The nominal life V_N is a useful parameter to quantify the time within 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 preservation can be implemented along the time. On the contrary, as far as Use and Human life (U) performance level are concerned, in particular PL3U (Life Safety), it is evident that the condition V_N<50 years corresponds to a annual rate of exceedance of the earthquake that is compatible with the PL greater than what demanded, and consequently a not acceptable annual rate of exceedance of the performance λ_{PLi} (even if not explicitly evaluated).

Moreover, it is worth noting that the employment of V_N is correct from a conceptual point of view only if a time-dependent hazard is available. PERPETUATE project did not deal with this topic but it is evident the use of time-dependent seismic hazard maps would be much advisable with the PBA procedure proposed in these guidelines.

In case 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 behaviour. The results of nonlinear analyses can help to single out the weaker elements and detect the irregular behaviours (torsional effects, irregularity in elevation); seismic preventive interventions should be focused to the aim of adjusting them, rather than proceeding to the spread and indiscriminate 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 usable life of the building is considered and further decisions are postponed. Another alternative is the revision of safety requirements that, 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 forecast in the future, when more accurate and validated tools will be available for sure, due to the improvements of applied research in the field.

Suggestions on possible strategies to be adopted for the selection of proper seismic strengthening interventions, taking into consideration conservation requirements, are provided in Annex (Recommendations P.C.M. 9/2/2011, 2011). Some traditional and innovative techniques are briefly described and their impact on the architectonic asset is judged.

The results achieved in this step are reported in Table 15.

Table 15. Results of step 8: rehabilitation decisions.

Outcome		Parameters	Deliverables
1	Return period compatible with the fulfilment of PLs	$T_{R,PLi}$	D35
2	Safety indexes (related to each PL)	$I_{S,PLi}$	D35
3	Nominal life (related to each PL)	$V_{N,PLi}$	D35
4	Design of strengthening intervention	techniques	D34

9. References

ASCE/SEI 41/06 (2007). Seismic Rehabilitation of Existing Buildings. American Society of Civil Engineers, Reston, VA.

Beyer K. (2012). Peak and residual strengths of brick masonry spandrels. *Engineering Structures*; 41: 533-547.

Blandon C.A., Priestley M.J.N. (2005). Equivalent viscous damping equations for direct displacement based design. *Journal of Earthquake Engineering*, Vol. 9, Special Issue 2, pp. 257-278.

Calvi G.M. (1999). A displacement-based approach for vulnerability evaluation of classes of buildings. *Journal of Earthquake Engineering*, vol. 3, no. 3, pp. 411–438.

Cattari S., Lagomarsino S. (2012). Perfomance-based approach to earthquake protection of masonry cultural heritage. *Structural Analysis of Historical Constructions* – Jerzy Jasieńko (ed) © 2012 DWE, Wrocław, Poland, ISSN 0860-2395, ISBN 978-83-7125-216-7, pp.2914-2922.

Cornell C. Allin, Jalayer F., Hamburger R. O., Foutch D.A. (2002). Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines, *Journal of Structural Engineering*, Vol. 128, No. 4, April 1, 2002, pp.526-533.

Eurocode 8 - Part 1-1 (2004). Design provisions for earthquake resistance of structures. Part 1-1: General rules – Seismic actions and general requirements for structures. ENV 1998-1, CEN: Brussels, 2004.

Faccioli E, Cauzzi C, Paolucci R, Vanini M, Villani M, Finazzi D (2007). Long period strong ground motion and its use as input to displacement based design. In K. Pitilakis (Editor), *Earthquake Geotechnical Engineering: 4th International Conference on Earthquake Geotechnical Engineering – Invited Lectures*

Fajfar, P. (2000). A non linear analysis method for performance-based seismic design. *Earthquake Spectra*, 16,3: pp.573-591.

Fajfar P. (1999). Capacity spectrum method based on inelastic spectra. *Earthquake Engineering and Structural Dynamics*. 1999; 28(9): 979-993.

Fajfar P., Dolšek M. (2012). A practice-oriented estimation of the failure probability of building structures. *Earthquake Engineering & Structural Dynamics*, Vol. 41, Issue 3, pp-531-547.

FEMA 440 (2005). Improvement of nonlinear static seismic analysis procedures. ATC: Washington.

Freeman S.A., Nicoletti J.P., Tyrell J.V. (1975). Evaluation of existing buildings for seismic risk- A case study of Puget Sound Naval Shipyard, Bremerton, Washington, *Proceedings of the U.S.National Conference on Earthquake Engineers,* EERI, Berkeley, pp.113-122.

Freeman SA. (1998). The capacity spectrum method as a tool for seismic design, In: *Proc. 11th European Conference of Earthquake Engineering*, Paris, France.

Priestley M.J.N., Calvi G.M., Kowalsky M.J. (2007). Displacement-based Seismic Design of Structures, IUSS Press, Pavia, Italy, 721 pp.

Recommendations P.C.M. 9/2/2011 (2011). Seismic assessment and risk mitigation of cultural heritage according to Italian Technical Code for Construtions (NTC 2008). G.U. n. 47 of 26-2-2011, Suppl. Ord. n. 54 (in Italian).

List of Deliverables:

Deliverable D4 (2010). Classification of the cultural heritage assets and description of the limit states and identification of damage measures. PERPETUATE Project.

Deliverable D5 (2010). Abacus of the most common seismic damage. PERPETUATE Project.

Deliverable D6 (2010). Review of innovative techniques for the knowledge of cultural assets (geometry, technologies, decay, …). PERPETUATE Project.

Deliverable D7 (2010). Review of existing models for global response and local mechanisms. PERPETUATE Project.

Deliverable D10 (2010). Characterization of the seismic hazard scenario for historical buildings. PERPETUATE Project.

Deliverable D11 (2011). Report on permanent displacement analysis of the building through remote sensing techniques. PERPETUATE Project.

Deliverable D12 (2012). Results of experimental test on damage measures and reference values to be considered. PERPETUATE Project.

Deliverable D13 (2011). Definition of demand spectra and other intensity measures for different soil categories and site condition. PERPETUATE Project.

Deliverable D14 (2011). Report on the SFI effects for ground shaking. PERPETUATE Project.

Deliverable D15 (2012). Results of laboratory and in-situ tests on masonry properties and table with mechanical parameters to be adopted in numerical modelling. PERPETUATE Project.

Deliverable D16 (2011). Report on microtremor measurement for structural identification. PERPETUATE Project.

Deliverable D17 (2012). Correlation of limit states and damage levels for types of buildings. PERPETUATE Project.

Deliverable D18 (2011). Results of in-situ microtremors surveys and array measurements at selected sites. PERPETUATE Project.

Deliverable D21 (2011). Report on the SFSI, foundation performance and vulnerability assessment for permanent ground displacement. PERPETUATE Project.

Deliverable D22 (2012). Definition of confidence factors for the safety verification. PERPETUATE Project.

Deliverable D23 (2012). Proposal of modelling strategies for artistic assets. PERPETUATE Project.

Deliverable D24 (2011). Report on vector-valued characterisation of seismic hazard with respect to strong-motion parameters. PERPETUATE Project.

Deliverable D25 (2011). Report on the development of the simplified soil-foundation model. PERPETUATE Project.

Deliverable D26 (2012). Modelling strategies for seismic global response of building and local mechanisms. PERPETUATE Project.

Deliverable D27 (2012). Formulation of vulnerability models including the survey form to collect the data required by the adopted models. PERPETUATE Project.

Deliverable D29 (2012). MB_PERPETUATE - A Macro-Block program for the seismic assessment (Freeware software for the safety verification of seismic local mechanisms). PERPETUATE Project.

Deliverable D33 (2012). Results of the shaking table and imposed displacements tests. PERPETUATE Project.

Deliverable D34 (2012). Results of experimental tests on strengthening techniques and guidelines for the design. PERPETUATE Project.

Deliverable D35 (2012). Definition of seismic safety verification procedures for historical buildings. PERPETUATE Project.

Deliverable D36 (2012). Definition of modelling and seismic safety verification procedures for artistic assets. PERPETUATE Project.

Deliverable D40 (2012). Final report on the application of the proposed methodology to the case studies selected. PERPETUATE Project.

ANNEX – INTERVENTION TECHNIQUES FOR THE SEISMIC PRESERVATION

A1. Strategies for the selection of interventions

Performance Based Assessment, performed according to PERPETUATE guidelines, gives a comprehensive description of the seismic behaviour of the building, which have to be complemented with historical knowledge, detailed survey and qualitative interpretation. Rehabilitation decisions and selection of interventions must fulfil with the goals of safety and durability, and should be limited in order to produce minimum impact on the historical building. It is useful to remember that periodic maintenance is the starting action to avoid material deterioration, prevent from the need of later major transformative interventions and may also help in the seismic preservation.

The selection of intervention techniques should be evaluated on a case-by-case basis, preferring the least invasive ones and those with the greatest compatibility to conservation criteria, taking into account the requirements of safety and durability. Precedence should be given to reversible interventions (as much as possible); new materials can be useful, if they are compatible with the original ones and do not affect their durability.

As much as possible, interventions should respect the original constructive techniques and structural concepts of the construction, as well as any significant transformations that may have occurred throughout the history of the building. From this point of view, the damaged structural elements should be repaired rather than substituted, whenever possible, and the deformations and alterations which testify its past should be maintained, eventually adopting measures to limit the negative effects on safety conditions.

Particular attention should be paid to the executive phase of the interventions, in order to check the accuracy of the hypothesis assumed in the design process and insure their real effectiveness. All realized works must be documented in detail, in order to be available for future generations, when further interventions will be necessary.

Sometimes historical buildings are subjected to interventions related to the update of technological systems: electrical, water and sanitary, heating, air conditioning, fire. Even if they are not considered structural intervention, they can be invasive and reduce the capacity of structural elements. A PBA is necessary when the impact on the construction is significant.

An intervention must also be evaluated considering its cost and comparing it to the benefits produced and the actual need.

In the following, some general indications for the choice of seismic strengthening interventions for masonry buildings are given, with reference to techniques widely used today. For each kind of pathology or vulnerability more than one intervention is possible, with distinct characteristics in terms of effectiveness, invasiveness, reversibility, compatibility, durability and cost.

The possibility of adopting provisional interventions (such as shorings) should not be overlooked, which due to their intrinsic reversibility appear interesting with respect to preservation and after careful evaluation, may also result to be the definitive solution for historic buildings.

The following indications must not be seen as a list of interventions to perform anywhere and everywhere, but only as possible solutions to adopt in cases where they are really needed, according to the outcome of the PBA. Obviously, techniques that have not been cited herein should not be excluded; in particular, innovative methods are promising, even if their effectiveness and impact on the cultural asset have to be carefully evaluated in advance.

A2. Interventions to improve connections

Such interventions are aimed to provide a global behaviour to the building, through the realisation of good clamping between walls and effective floor to wall connections. Moreover, eventual thrusts due to arched structures or roof rafters must be verified, making sure the connections are sufficient to balance them. The realisation of these interventions is an essential prerequisite for applying global seismic analysis methods to the building, which are based on the in-plane behaviour of the masonry walls, assuming their stability with respect to out-of-plane seismic actions.

The insertion of tie-rods, whether in steel or other materials, placed in correspondence to bearing walls at the level of floor diaphragms, can aid the behaviour of the whole building, since it confers good connection between orthogonal walls and supply an effective constraint against out-of-plane overturning of the masonry panels, when stability is not guaranteed by floors or other structures. Moreover, the insertion of tie-rods improves in-plane behaviour of walls with openings, because it increases the seismic capacity of masonry spandrels. Simple bolts or slabs may be utilised to anchor tie-rods to walls; with the exception of particularly poor masonry walls, made up of small sized elements, the use of bolts to plates is preferable due to the fact they bear a larger wall area. In any case, the dimensions of the end-contraints must be chosen on the basis of the characteristics of the masonry elements (panels, columns, pillars) to which they are connected. Often local consolidation of masonry is necessary in the anchoring zone. It is not advisable to place the end-constraints into the wall thickness, especially for multi-leaf masonry. Utilising anchoring for adherence (through injections of resin or cement-based mortar into the masonry) should be carefully considered, due to both possible incompatibility of the materials as well as the difficulty of controlling the effectiveness of the anchoring itself. The tie-rods must generally be positioned next to principal load bearing walls; if it is necessary to core longitudinally the wall, tie-rods should not be injected, in order to render the intervention reversible and allow an eventual re-tensioning. As far as the tensioning of the rods is concerned, limited tension should be used in order to induce in masonry limited compressive stresses, well below strength values.

External ties with metallic or composite materials can guarantee an effective connection between orthogonal walls when dealing with smaller buildings, where the length of the sides of the ties is not too high, or when additional anchoring are provided in correspondence with internal intermediate masonry walls. It is necessary to avoid the concentrated insurgence of tension near sharp masonry corners, for example with opportune elements of diffusion; when strips made of composite materials are used, the angles must be smoothed.

Clamping between adjacent parts or intersecting walls (*cuci-scuci* technique) can be used (with stone or brick elements) whenever the junctions between masonry elements are deteriorated (due to cracking) or particularly decayed. These interventions are destructive and substitutive of ancient materials by their very nature, and therefore should be used with caution, on a case-by-case basis.

The use of reinforced bars perforations must be limited to cases where there are no other feasible solutions due to the high degree of invasiveness of such elements and their dubious effectiveness, especially in the presence of multi-leaf masonry panels. The durability of the inserted bars must be guaranteed (stainless steel, composite materials, etc.) as well as the compatibility of the injected cement.

It should be noted that these local junctures are effective for the overall behaviour of the building only when good masonry walls are present. When masonry is of poor quality, the use of tie-rods is preferred as it guarantees a better connection.

Stringcourses (or tie-beams) at roof level can provide an effective solution for joining walls in zones where the masonry is less cohesive due to limited levels of vertical compression, and to improve the connection with the roof coverings. They can be realised in the following ways:

- In reinforced masonry, which permits connection through a technique that provides maximum preservation of the characteristics of the existing masonry. In fact, they must be realised with masonry of good characteristics, usually with solid bricks and hydraulic mortar; in a small inner core, steel or composite bars are placed and made adherent to the masonry by concrete. The connection between the tie-beam and the underlying masonry wall usually is simply guaranteed by a good adherence, the friction and the irregularity of the contact surface (in some cases it may be advisable to consolidate the tops part of the masonry wall by mortar injections). The use of inclined reinforced bar perforations should be avoided whenever possible.

- In steel, which represents a valid alternative due to its lightness and limited invasiveness. They can be realized in two ways:

a) by way of a reticular truss structure, placed horizontally on the top surface of the masonry wall, made of steel angular profiles or plates which are welded together and connected by way of pseudo-vertical reinforced bars perforations;
b) by way of plates or profiles at the two surfaces of masonry wall, placed just beneath the top and connected by pass-through bars. In the presence of poor quality masonry, this intervention must be joined with consolidation mortar injections to improve the masonry portions involved. Metallic stringcourses are also particularly suitable for connecting wooden rafters of the roof and contribute to the balance of eventual thrusts.

- In reinforced concrete (r.c.), only if the height of the tie-beam is limited, to avoid excessive weight and stiffness, which has proven to be negative due to the generation of shear stresses between r.c. and masonry, with consequent sliding and crumbling of the latter. In particular, such effects are manifested when the roof covering has also been stiffened and weighed down by a r.c. slab down. In general, it is advisable to consolidate the tops of the masonry in order to limit the different stiffness of the two elements. The connection with reinforced perforations may be adopted only when considered really necessary and after having verified that bars can be anchored efficaciously to the eventually consolidated masonry.

The effectiveness of floor-to-wall and roof-to-wall connection is necessary to avoid beams pull-out, which causes ceiling collapse, and contribute to the box-type behaviour. Moreover, these connections permit the horizontal diaphragms to better distribute seismic actions among vertical elements. In the case of intermediate diaphragms (at floor levels), the ends of the wooden beams can be anchored to the masonry through metallic elements or other materials that are resistant to traction, anchored to the opposite wall surfaces.

The insertion of stringcourses in r.c. into the thickness of the walls at the intermediate levels has a negative impact on the seismic behaviour of the walls, besides of being incompatible with conservation criteria. Eventually, in the case of walls that are very flexible out-of-plane, due to the high distance between orthogonal bearing walls, steel plates stringcourses may prove useful; they are made with plates or profiles on two surfaces, connected by way of pass-through bars. They furnish a significant flexional out-of-plane stiffness to the walls and prevent from the development of cracking mechanisms to the masonry spandrels. In the case of external walls, the effectiveness of such stringcourses with only one internal profile, anchored to the external masonry leaf by way of passive anchored bar must be verified carefully.

A3. Interventions to reduce thrusts of masonry vaults and their strengthening

Strengthening interventions to arched or vaulted structures may be realised by utilising the traditional technique of tie-rods, which balance the thrusts induced on the vertical walls and prevent from the movement of the springs. The tie-rods are normally placed at the level of haunches; whenever this position is not possible, the tie-rods can be connected at diverse levels (e.g. the extrados) as long as the shear and bending stresses induced to the wall has been verified. The tie-rods must be put in place with adequate pre-solicitation to absorb part of the thrust action, estimated by way of calculations (an excessive traction may cause localised damage).

In order to balance the thrusts of vaulted structures, buttresses or masonry wall thickening can be considered. These interventions has an appearance impact on the construction and effectiveness is subject to the creation of good clamping of these added elements to the existing masonry wall; to this end, discrete connections with stone or brick elements can be placed and it is also necessary to create an adequate foundation.

The extrados realisation of a concrete cap, whether reinforced or not, should be avoided for the reduction of the state of compression in the masonry vault and the increase in seismic masses, besides of the loss in terms of cultural value in the historic building.

It is possible to apply at the extrados stripes of composite materials (FRP) because it is lighter and also removable technique of intervention. Nevertheless, the following issues should be considered: a) differences in transpiration that may occur in the areas where FRP are glued (critical in case of presence of frescoes); b) durability (the experience of the behaviour over time, both of the fibres as well as the resin used as glues, is still rather limited); c) incomplete reversibility (the superficial parts of the masonry remain impregnated with resin). The position of the FRP stripes, especially in the presence of complex vaults, should be defined on the basis of an accurate structural analysis that demonstrates its effectiveness. The application of FRP stripes at the intrados is effective only when coupled with the realisation of sub-arches, made of masonry, steel or other materials (e.g. in-situ glued laminated timber), which are able to balance the peeling phenomenon. As an alternative, through-thickness anchoring regularly spaced along the intrados can be used, but this solution is nevertheless very invasive.

In order to reduce thrusts, it is possible to intervene by reducing the extrados loads (filling materials can be lightened), but it is necessary to pay attention to the alteration of the original thrust line in the arch. Moreover, it is worth noting that permanent loads render the vault less sensitive to accidental loads. To this end, the use of a light conglomerate can be an effective solution, because it contributes to prevent from the formation of collapse mechanisms; however, the mechanical properties of this filling material must be very poor, in order to assure reversibility and allow future interventions.

Obviously, in the presence of cracks, repair must be made in order to restore the original contact between the ashlars (or in mortar joints) by way of local mortar injections, that can be realized both from the extrados and the intrados. In particular cases, wedges can be utilised, in order to force the contact between disconnected elements.

Vault portions in which crushing of masonry occurred must be substituted. Particular attention must be paid in cases where significant loss of the original curvature of the arch or vault is present. Its recovery is often problematic, thus a sub-arch or other integrative structures can be adopted. An interesting alternative solution is the positioning at the extrados of not-injected cables, which after a post-tensioning transfer radial actions which modify the trust line so that it were closer to the middle line of the arch; this intervention is also able to recover a little bit the original curvature.

A4. Interventions to reduce the flexibility of horizontal diaphragms

The floor diaphragms must be effectively connected to the masonry walls by means of a sufficient support length and connection elements that impede the pull-out of timber beams or steel profiles. The role of diaphragms in seismic behaviour of masonry buildings is to transfer lateral actions to the walls which are parallel to the direction of the earthquake. Moreover, they improve the constraints for the walls loaded out-of-plane. To this end, it is usually sufficient a limited in-plane stiffening of the horizontal diaphragms (as can be demonstrated by equivalent frame modelling of buildings), that must be realized without increasing the dead loads. Exaggerated stiffening, besides of the increase of seismic masses, tends to concentrate forces into few stiffer walls, usually limiting the displacement capacity at failure; moreover, in case of plan irregularity, torsional effects are increased and the exterior walls turn out to be overloaded. Compatible to the above goals, it is opportune that wooden diaphragms are preserved as much as possible, due to their lightness, with simple intervention.

A limited stiffening of wooden floor diaphragms can be performed at the extrados on the planks. One can set a second wooden plank over an existing one, which should be posed in a orthogonal or slanted direction and by paying particular attention to junctions with lateral walls. An alternative or addition is to use diagonal reinforcements with steel plates or composite material stripes (FRP), which are fixed to the original plank. An analogous benefit can be obtained by diagonal bracing made of steel bars of cables. In the case of ordinary wooden floors, the junction between to parallel walls to the beams should be done carefully, for example by putting fixed bands to the planking and anchoring them to the masonry.

When strengthening of floor diaphragms is necessary for static vertical actions, with the wood-wood technique it is possible to limit the flexional deformability and increase resistance with a second layer of planking, placed orthogonally to the existing planking. The new continuous planks are connected to the beams by way of pins (even in timber).

Another reinforcement technique is the realization over the plank of a thin reinforced concrete slab (eventually with a lightened material), collaborating with the original timber beams through proper connectors. The effects of this type of intervention should be evaluated in relation to specific conservation requirements.

When the wooden elements are not adequately connected to the masonry, it is necessary to link the floor diaphragm to the walls by way of regularly placed elements.

In cases where diaphragms with steel profiles are present, with interposed arched solid bricks or hollow-clay tiles, it may be necessary to connect them by way of transverse metallic bands, welded to either the intrados or the extrados.

A5. Interventions on the roof covering structure

The original wooden roof has to be maintained, not only because of the conservation of its cultural value but also due to its flexibility, which is compatible with that of the system of orthogonal masonry walls, and its lightness, which limits the seismic actions just in the highest parts of the building.

Usually, links and connections among timber elements should be improved, as well as those with the top of the masonry wall; technological details should be compatible and similar to the original historical constructive details, when effective. The ends of timber rafters and king trusses can be connected to steel plates anchored to the walls; if a stringcourse (tie-beam) is present, these connections are very easy to be realized.

Whenever roofs produce thrusts to the perimeter masonry walls, if possible it is better to implement the structural configuration of the original roof in order to eliminate these thrusts, by adding steel and/or timber elements. Otherwise, a tie-beam stiff enough must be added on the top of the wall, in order to balance these thrusts.

In the cases of timber trusses, good connections in joints must be present, which are necessary for avoiding shifting and breakage during the seismic action. This can be improved by plates and metal bars, or with other materials (e.g. FRP).

If the roof is not stiff enough in the sloped diaphragms, some improvement may be useful, on condition that the total weight is not increased and the structure remains not totally rigid. Technical solutions are similar to those introduced in case of horizontal timber floors (double timber planks, FRP, diagonal steel plates). A proper system of bracing, by crossed post-tensioned bars, can be introduced at the intrados.

A6. Interventions for increasing the strength of masonry panels

These interventions are aimed at both repairing of deteriorated and damaged masonry and improving the mechanical properties of the masonry. The technical solutions applied should be evaluated also on the basis of the typology and the quality of the masonry. The interventions must utilise materials with physical-chemical and mechanical characteristics analogous with or at the very least compatible with the building existing materials. The interventions should aim to provide a substantially uniform resistance and stiffness to the walls, which can also be accomplished by the improvement of clamping between walls, whenever needed. The insertion of materials diverse from the existing masonry, and particularly cement-base mortars, should be utilised with caution and only where the cost-benefit ratio (with reference to safety and conservation) is favourable (e.g. when the negative impact of cement is minor than the loss connected to the need of demolishing and replacing elements).

Depending on the case one can:

- Repair at local level cracks or deteriorated masonry portions;
- Reconstruct masonry portions in correspondence to recesses, unused chimney flues, badly closed openings;
- Improve the characteristics of the particularly poor masonry types, which can be inadequate due to quality of mortar or size and assembling of blocks.

Cuci-Scuci intervention consists of disassembling masonry blocks in the deteriorated portions (e.g. along a crack) and reassembling masonry again, with the aim of bringing back continuity. The use of materials that are similar to the original ones in shape, size, stiffness and resistance is advised. The new elements should be connected to the existing masonry with adequate clamping in the external leaves and, when possible, in the transverse section, in order to provide maximum homogeneity and monolithic nature to the repaired wall. Such interventions can also be utilised for closing recesses and unused chimney flues, particularly when they are positioned near corners or at the intersection between masonry walls.

The use of mortar injections is aimed at improving the mechanical characteristics of masonry that needs strengthening. This technique cannot provide adequate clamping between the walls. Such an intervention proves to be ineffective if employed on types of walls that by their very nature are not easy to be injected (scarce presence of voids and lack of interconnections among them). Particular attention should be paid to the choice of the injecting pressure of the mix, in order to avoid the insurgence of transverse dilatation. Particular care should be given to the choice of the mixture to be injected, evaluating the chemical-physical-mechanical compatibility with the masonry under repair. Cement-based mortars may damage the walls and especially the wall surfaces, due to salt production. The emergence of soluble salts in the mortar creates crusting on the wall surface, which is particularly damaging in the presence of ancient paintings, mosaics or frescoes. Such mortar should be used only after having accurately evaluated any eventual negative effects.

Interventions of repointing of mortar joints, when applied deeply on both sides of the walls, can improve the mechanical characteristics of the masonry, especially when the walls are not particularly thick. When used with medium or thick walls, in the cases where the external leaves are not well connected together, such an intervention is not sufficient to guarantee a consistent increase in resistance and it is therefore advisable to perform this intervention in combination with other strengthening techniques. Moreover, it is worth noting that this technique can cancel significant historic traces (original manufacturing of ancient masons) and, in case of exposed stone masonry, it can modify significantly the visual perception.

Quite recently some variations of the above-mentioned techniques have been proposed, all ascribable to the idea of reinforced repointing. In case of solid brick masonry, a thin bar (in stainless steel or composite materials) can be placed inside the joint, after scarification and before repointing. As far as irregular stone masonry is concerned, a thin flexible cable can be inserted along joints, without a regular meshing, before repointing (Reticulatus); some experimental tests proved the technique is able to provide some improvement of masonry properties.

The insertion of artificial transversal elements (diatones) is aimed at providing or improving the transversal connection between external leaves of a masonry wall, avoiding their separation due to instability phenomena under compression. Moreover, such interventions give the walls monolithic behaviour in cases of out-of-plane seismic actions. The intervention consists in coring transversally the wall (diameter around 100 mm), put a light reinforcement and inject cement or hydraulic mortar, in order to obtain a stiff element, which connect the external leaves by friction. Different technological details can be adopted (e.g. coring at smaller diameter and injecting cement-mortar in a proper sock, in order to prevent from diffusion in the masonry).

In cases where a small portion of masonry needs to be reinforced, because of local bulging, a valid solution is the use of anti-expulsion bars, made of thin transverse bars bolted with small flat washers to the external leaves of the wall. The scarce invasiveness attributed to this intervention renders it ideal in cases of wall leaves separation and, due to the possibility of post-tensioning the bar, a quote of deformation can be recovered. It is worth noting that bars must not be injected, in order to assure the maximum reversibility. This technique requires that masonry external leaf is not too irregular and of poor quality.

New techniques have been proposed quite recently, aimed at connecting the wall in the transverse direction and improving the monolithic behaviour. CAM system is a stitching method made by a regular mesh of thin steel bands, which pass the masonry through the thickness and tie block together. Ticorapsimo system is another stitching method made by thin cables, made of basalt composite fibres. It should be noted that these interventions may be effective in case of poor masonry but can be also quite invasive, even in relation to the extensions necessary for their effectiveness, and for these reasons they should be applied only when absolutely necessary.

The reinforced concrete jacketing of the masonry constitutes an invasive intervention and is not coherent with conservation principals: it consists in the application of steel meshes of the two sides of masonry wall, connected transverse bar and covered by a concrete layer of 4 to 6 cm (usually shotcrete is used). The strength improvement is obtained only if the technique is applied correctly (indeed, sometimes jacketing is applied only on one side or the transverse bars are omitted, because in stone masonry it is not simple to drill). From a seismic point of view, it is opportune to consider that it increase very much the stiffness of masonry panels; for this reason the seismic behaviour of the building is strongly modified, not always in the positive direction (as the global displacement capacity is limited). Such technique can be used only in cases where the masonry is greatly damaged or incoherent and it is impossible to intervene with other methods; in any case, it is admissible not as a widespread intervention but only in limited portions of the masonry. In these cases, an alternative may also be the local demolition and reconstruction of that portion of masonry.

Jacketing with composite meshes (GFRP) is a recently proposed alternative, in which hydraulic mortar may be used (but usually cement concrete is preferred) and the thickness of the covering layers is smaller. The result is a lower increase of stiffness and masses, but the intervention still remain invasive and not advisable for a wide application in cultural heritage assets.

A very effective solution for the seismic strengthening of masonry panels is the application of the wall surfaces composite stripes (CFRP), glued with epoxy based resin. Stripes are usually disposed in two orthogonal directions; the adoption of a vertical and horizontal disposition (with horizontal stripes placed over the vertical ones, with the aim of acting like stirrups) is preferable in comparison with diagonal solutions.

In D34 interesting experimental results are presented concerning the effectiveness in masonry panels of Horizontal Narrow CFRP stripes. The use of stripes only in the horizontal direction reduces a little bit the shear strength but increases very much the ductility; moreover, the application of the method is mush simpler than that with also vertical stripes, due to difficulty of gluing and anchoring them in real cases. Tests showed that even very narrow stripes are sufficient to guarantee the effectiveness, so the invasiveness of the intervention is acceptable. The high ductility of strengthened panels is very positive for the seismic performance of a masonry building, according to the concept of increasing displacement capacity rather than strength.

The insertion inside masonry walls of post-tensioned vertical tie-rods is applicable only in specific cases and when the masonry has been proven to be able to support the increase in vertical load. In any case, the loss along time of the initial applied tension, caused by the long-term deformation of masonry, must be taken into consideration. Such a solution tends to modify the original behaviour of masonry constructions, in which masonry walls has no tensile strength and the stability is assured by the geometry; under this light, the method must be considered quite invasive and is advisable only in the absence of other alternatives.

In the case of decorated walls and frescoed surfaces, strengthening interventions cited herein can be utilised only with extreme caution, with the aid of experts who are specialised in the restoration of such surfaces. When possible, it is better to strengthen contiguous walls, with interventions of similar effectiveness, trying to minimize the drift demand in the decorated panels, by means of solutions that can be checked through proper models.