# Guidelines for evaluation and mitigation of seismic risk to cultural heritage

**Luglio 2006**

Il presente documento trae origine da un'intesa istituzionale tra il Dipartimento della Protezione Civile e il Dipartimento per i Beni Culturali e Paesaggistici (Direzione Generale per i Beni Architettonici e Paesaggistici del Ministero per i Beni e le Attività Culturali), in attuazione del Decreto Interministeriale 23 maggio 2005, finalizzato all'elaborazione delle Linee Guida per l'applicazione al patrimonio culturale della normativa tecnica di cui all'Ordinanza della Presidenza del Consiglio dei Ministri 20 marzo 2003, n. 3274. Il documento, elaborato dal Gruppo di Lavoro istituito con il Decreto Interministeriale, è stato trasmesso per un parere al Consiglio Superiore dei Lavori Pubblici, del Ministero delle Infrastrutture e dei Trasporti, che ha formato una Commissione Istruttoria, la quale lo ha esaminato ed ha apportato alcune significative modifiche. Questo documento costituisce il testo emendato, allegato al parere n. 66 dell'Assemblea Generale del Consiglio Superiore dei LL.PP., reso nella seduta del 21 luglio 2006. Esso viene reso pubblico per favorire la diffusione delle conoscenze in materia di rischio sismico dei beni culturali, in attesa di una formale adozione.

## GRUPPO DI LAVORO

ai sensi dell'art. 3 dell'Ordinanza P.C.M. 3431/2005, istituito con decreto interministeriale del 23 maggio 2005

Arch. Roberto CECCHI (coordinatore - Direttore generale per i beni ambientali e paesaggistici, Min. per i beni e le attività culturali)
Prof. Michele CALVI (coordinatore - ordinario di Tecnica delle costruzioni, Università degli Studi di Pavia)
Ing. Agostino GORETTI (redazione documento e segreteria - Dipartimento della Protezione Civile)
Prof. Paolo FACCIO (redazione documento - Associato di Restauro architettonico, IUAV di Venezia)
Prof. Sergio LAGOMARSINO (redazione documento - ordinario di Tecnica delle costruzioni, Università di Genova)
Prof. Antonio BORRI (ordinario di Scienza delle costruzioni, Università degli Studi di Perugia)
Prof. Giovanni CARBONARA (ordinario di Restauro architettonico, Università degli Studi di Roma "La Sapienza")
Prof. Giorgio CROCI (ordinario di Tecnica delle costruzioni, Università degli Studi di Roma "La Sapienza")
Prof. Michele JAMIOLKOWSKY (ordinario di Geotecnica, Politecnico di Torino)
Prof. Gaetano MANFREDI (ordinario di Tecnica delle costruzioni, Università degli Studi di Napoli "Federico II")
Ing. Luciano MARCHETTI (Direttore regionale per i beni culturali e paesaggistici del Lazio, Min. per i beni e le attività culturali)
Prof. Claudio MODENA (ordinario di Tecnica delle costruzioni, Università degli Studi di Padova)
Prof. Paolo ROCCHI (ordinario di Restauro architettonico, Università degli Studi di Roma "La Sapienza")
Prof. Carlo VIGGIANI (ordinario di Geotecnica, Università degli Studi di Napoli "Federico II")

con il contributo del Prof. Carlo BLASI (ordinario di Restauro architettonico, Università degli Studi di Parma)

## COMMISSIONE ISTRUTTORIA

istituita dal Presidente del Consiglio Superiore dei LL.PP. con lettera di nomina del 30 marzo 2006 (prot. 1287)

Ing. Valentino CHIUMARULO (primo relatore, presidente della 3^ Sez. Cons. Sup. LL.PP.)
Ing. Giovanni GUGLIELMI (presidente della 1^ Sez. Cons. Sup. LL.PP.)
Prof. Ing. Silvio ALBANESI (esperto ex art. 7, Legge n. 1460/1942)
Ing. Pietro BARATONO (esperto ex art. 7, Legge n. 1460/1942)
Arch. Marisa BONFATTI PAINI (componente dell'Ass. Gen. del Cons. Sup. LL.PP.)
Prof. Ing. Franco BRAGA (esperto ex art. 7, Legge n. 1460/1942)
Prof. Ing. Alberto CASTELLANI (componente dell'Ass. Gen. del Cons. Sup. LL.PP.)
Arch. Roberto CECCHI (componente dell'Ass. Gen. del Cons. Sup. LL.PP.)
Arch. Elisabetta D'ANTONIO (componente dell'Ass. Gen. del Cons. Sup. LL.PP.)
Ing. Giacomo DI PASQUALE (esperto ex art. 7, Legge n. 1460/1942)
Prof. Ing. Giocarlo IMBRIGHI (esperto ex art. 7, Legge n. 1460/1942)
Ing. Giovanni MANIERI (Regione Emilia Romagna)
Prof. Ing. Claudio MODENA (esperto ex art. 7, Legge n. 1460/1942)
Dott.ssa M. Maddalena LA MONTAGNA (componente dell'Ass. Gen. del Cons. Sup. LL.PP.)
Avv. Aldo LINGUITI (Avvocato dello Stato)
Prof. Paolo ROCCHI (esperto ex art. 7, Legge n. 1460/1942)
Prof. Alberto PRESTININZI (componente dell'Ass. Gen. del Cons. Sup. LL.PP.)
Ing. Giancarlo SANTARIGA (componente dell'Ass. Gen. del Cons. Sup. LL.PP.)

con il contributo del prof. Paolo FACCIO, del prof. Sergio LAGOMARSINO e dell'ing. Aniello VIETRO

**TABLE OF CONTENTS**

# 1 GUIDELINE OBJECTIVES

## 1.1 Objectives and criteria

These guidelines give indications for evaluating and reducing seismic risk for protected cultural heritage, with reference to technical construction regulations[1] (hereon called NTC), of which Minister's Decree dated 14 September 2005, and particularly to Annex 2 of the "Technical regulations for designing, evaluating and seismic upgrading of buildings", of the P.C.M. Ordinance no. 3274/03 and its subsequent modification and integrations[2] (aforesaid Annex 2 shall be called "Ordinance" from hereon).
The "Cultural Heritage and Landscapes Code" (from hereon "Code"), Decreed law no. 42 dated 22 January 2004, established in Article 4 that the functions of protecting national cultural landmarks attributed to the State and carried out by the Ministry of Cultural Heritage and Activities; as already recognised by Article 16 of Decreed Law no. 64 dated 2 February 1974 (provisions for construction with particular regulations for seismic areas). As far as interventions on historic landmarks are concerned, Article 29 of the Code, paragraph 4 states that for building landmarks situated in areas of high seismic risk, restoration must also include structural strengthening interventions. As defined by the Ministry in paragraph 5, these building codes shall also be applied for regional work tenders and in collaboration with universities and competent research institutions, who shall establish guidelines, technical regulations, criteria and models for strengthening interventions regarding the conservation of the cultural heritage.
The present Guidelines have been established with the intention of specifying a path of knowledge, the evaluation of seismic safety and the design of the eventual strengthening interventions that are analogous to regulations for unprotected buildings, but appropriately adapted to the needs and peculiarities of national cultural heritage. Our objective is to formulate, in the most objective manner possible, a final judgement on the safety and conservation guaranteed by seismic strengthening interventions. In particular, this document refers to masonry buildings.
Regarding the conservation of cultural heritage taking into account the seismic safety, it is necessary to provide analysis tools at various levels of depth. This should be applied to two diverse scales: the evaluation of the vulnerability of cultural heritage by territorial scale; the evaluation of the vulnerability of the safety and the design of strengthening interventions on individual buildings.
In order to apply these concepts, this document refers to methods that should not be interpreted as binding. Moreover, considering the continual evolution of the field, the guidelines should be adjourned regularly.

## 1.2 Guidelines contents

The various chapters of this document give indications for defining seismic hazard, in relation to the building site in question and its end use, and its structural capacity, by correct knowledge and modelling of the structure.
In chapter 2, safety requirements are suggested which are adequate for architectural landmarks of historic and artistic value. It is advisable to redefine the reference limits which do not refer only to the necessity of safeguarding people (ultimate limit state) and functionality (limit of damage state), but also to the loss of the structure and to damage to the artistic assets contained in the building as well. Moreover, levels of seismic protection have been suggested in relation to conservation needs and condition of use.
In Chapter 3, indications are given for the accurate definition of seismic activity which was found to be particularly useful when comparing structural collapse ground acceleration and that awaited in the site. Although this knowledge does not having a binding value, it contributes to the judgement of the level of risk of the building. The subdivision of Italian territories into zones and attributing a prefixed value of seismic activity to each represents a simple and effective scheme for designing new structures, but may lead to overestimates which may result to be critical for the conservation of existing buildings. Seismic activity, therefore, may be evaluated as to benefit from the available research that defines seismic danger in the

[1] Ordinary Supplement no. 159 of the Official Gazette no. 222 dated 23.9.2005.
[2] Ordinary Supplement no.. 72 of the Official Gazette no. 105 dated 8.5.2003, Official Gazette no.236 dated 10.10.2003 and S.O. no. 85 Official Gazette no.107 dated 10.5.2005.

Italian territory. When necessary, an in depth investigation on the local characteristics of the site may be effectuated (microzonation).
Knowledge of the building (Chapter 4) must be acquired by considering the indications outlined in point 11.5 of the Ordinance in conformity with the monitoring program of the state of conservation of protected cultural heritage (Attachment A) elaborated by the Ministry of Cultural Heritage and Activities – General Direction for Architectural landmarks and landscapes and finalised for the acquisition of knowledge about Italy's cultural patrimony.
In Chapter 5, diverse possibilities are illustrated of structural modelling of historic masonry buildings. In particular, for the evaluation of seismic safety, examples of three different levels with increasing complexity are identified respectively: LV1) for the evaluation of seismic safety to be effectuated over the national territory for all protected historic buildings[3]; LV2) for the evaluations to be adopted in the presence of localised interventions on limited areas of the structure; LV3) for the design of strengthening interventions which will modify the ascertained structural functioning or when an accurate evaluation of the seismic safety of the building is requested.
Finally, in Chapter 6, guidelines are described for seismic improvements, in other words, the reduction of ascertained vulnerability on the basis of results of the modelling and observation of any eventual damage. For each matter, possible interventions are also indicated which are critically examined in relation to their effectiveness, their impact on the conservation of the building (non-invasiveness, reversibility, and durability), and costs.

## 2 SAFETY AND CONSERVATION REQUIREMENTS

### 2.1 Tools for evaluating seismic safety on territorial scale

Putting mitigation policies into practice requires the knowledge of seismic risk of existing structures on a territorial scale.
To meet this need, with the intention of acquiring homogeneous and accurate knowledge of the risk to national treasures in the least amount of time possible, the General Direction for Architectural landmarks and landscapes of the Ministry of Cultural Heritage and Activities has elaborated a program for monitoring the state of conservation of protected cultural heritage. It consists in the creation of a databank of protected architectural landmarks that includes a series of data for each structure relative to the knowledge of the construction and the state of conservation, to the evaluation of its vulnerability and risk as well as any eventual design for preventative strengthening interventions. In Attachment A, the methods and structure used in this databank have been described.
The aim is to acquire in a reasonably brief amount of time, knowledge of the safety level of these buildings in high seismicity areas. Considering the elevated number of protected structures, in the case of extensive verifications on a territorial scale, these must be interpreted as an evaluation of seismic safety to be followed with simplified methodologies differing from those used for intervention projects. In any case, it is necessary to quantitatively evaluate collapse acceleration and compare it to the expected time of the site in a prefixed time interval and with a pre-established probability of exceeding this limit (expected acceleration of the site). This parameter that shall be defined in the seismic safety index below, is useful for highlighting the most critical situations and establish priorities for future interventions. If the collapse acceleration results to be significantly inferior to the expected limit of the site, this simply indicates the need to perform a more accurate evaluation and eventually intervene at a later date. (In fact, in tune with the probabilistic concept of safety, the structure may be considered safe with regards to an earthquake with

[3] The Ordinance no. 3274/03, in article 2, paragraph 3, states that within 5 years seismic tests must be performed on both buildings of strategic interest and of the infrastructural works in which their functionality during seismic events assumes a fundamental importance for the objectives of civil protection, as well as for buildings and infrastructural works that assume importance as the consequence of an eventual collapse. The P.C.M. Decree dated 21 October 2003 (Official Gazette no. 252 dated 29.10.2003) lists in the details for structural typologies of national competence as defined above: many structures belonging to the first category are protected buildings and, in the range of the second category are explicitly indicated as "buildings in which collapse may determine significant damage to historic, artistic and cultural patrimony (i.e. museums, libraries, and churches). It is evident that such verifications must be effectuated on nearly all protected landmarks.

a shorter return period than that of the expected acceleration and thus having a briefer expected acceleration rate).
In this document, certain simple mechanical models have been identified which are useful for the most diffuse types of historic structures; the adoption of these models though affected by uncertainties, has the ability of supplying a homogeneous evaluation on a territorial scale and thus is significant for the aim of designing future strengthening interventions.
The LV1 level allows the evaluation of collapse acceleration by means of a simplified method based on a limited number of geometric and mechanical parameters or which utilises qualitative tools (visual in situ investigation, recording construction characteristics, critical and stratigraphic surveys).
For more accurate evaluations of the single buildings, the tools to be adopted for the design of strengthening interventions are defined according to the level called LV3.

## 2.2 Criteria for evaluating seismic safety and effectiveness of strengthening inventions

For protected cultural heritage it is compulsory to adopt, when necessary, seismic improvement interventions instead of seismic upgrade. The term seismic improvement is intended to mean the execution of work able to give a building greater safety with respect to earthquakes, although the level of seismic protection may not necessarily be equal to that required for new constructions.
In the case of existing architectural and cultural buildings of historic interest, the obvious difficulty is to define a procedure to verify safety requirements which is analogous to those applied to ordinary buildings because of their typological variety and specific singularity as “monuments” (also due to the history of each building) which cannot be indicated with a univocal and trustworthy strategy of modelling and analysis. In these evaluations, one often meets both uncertainties in the behavioural model as well as in the parameters of the model itself. Moreover, as far as the interventions are concerned, it is not always possible to precisely quantify their real effectiveness and it is impossible to consider in a purely quantitative procedure the necessary conservation measures nor derive what is often useful to ascertain a higher level of seismic risk than that of ordinary buildings rather than intervene in an opposite way to the conservation criteria.
Nevertheless, it is necessary to calculate ground acceleration levels which correspond to the reaching of each limit provided for that type of building structure, in the previous situation and subsequently to the eventual strengthening intervention. In this, it is implicit that the knowledge of modelling and verification provisions for ordinary buildings cannot always be applied to historic buildings. However, it is necessary to proceed with an evaluation of seismic behaviour of the total structure with proper models.
In these guidelines, a path of knowledge and analysis is indicated in which judgement on the level of risk for a structure or the validity of an intervention emerges from the comparison of structural capacity and seismic hazard. Below, a qualitative and quantitative knowledge of building is evaluated and seismic activity, opportunely modulated in function of reasonable levels of seismic protection. Such a comparison is not meant as a compulsory verification in which capacity must result superior to the demand subsequent to the seismic action, but as a quantitative element to take into account along with others in a qualitative judgement of the whole which considers the needs of conservation, the willingness to preserve the building from seismic damage and safety requirements in relation to the fruition and function performed. In this context, all of the coefficients and parameters indicated in the following guidelines do not have prescriptive value.
In summary, for protected cultural heritage, it is possible to derogate respect for the updating from an operational point of view a possible procedure to be applied to the concepts expressed and the following:

- The evaluation of the index of seismic safety in present conditions (ascertained functionality): in this phase, a qualitative evaluation should be taken into account also for situations of known vulnerability but difficult to quantify.
- The evaluation of the index of seismic safety to which the structure can be upgraded with interventions that are compatible with the necessities of protecting singular specific characteristics:
  a) if the reachable seismic safety index, which takes into consideration the site hazard and the proposed end use, is compatible (see. tab. 2.1), the strengthening intervention is fully satisfying from a safety point of view as well, evaluated by means of a quantitative procedure;
  b) if the reachable seismic safety index is inferior to what is desired, or if interventions are necessary that would be too invasive, the designer must justify the intervention by qualitative means which must be shown to be explicitly relevant to the choices made and integrative to the relative

calculations. In particular, two diverse situations can be verified that can be justified by the relations below:

- the mechanical model even when developed with the most accurate analysis instruments is nevertheless inadequate for describing the complexity of the behaviour of a historic building in a reliable way: in this case the safety index can be fully satisfactory followed by intervention after the qualitative evaluation of the structural behaviour based on observations of the construction and historical analysis;
- the mechanical model is reliable, but specific conservation needs exist which impede the adoption of interventions which would be necessary to completely satisfy the seismic safety requirements: in this case one can, with the principals of seismic improvement, coherently and knowingly adopt systems of interventions which are less effective in terms of safety, but compatible with the needs of conservation, to avoid certain damage in cultural terms produced by invasive interventions.[4]

The goal is to avoid unneeded interventions, thereby favouring the criteria of minimal intervention, but also highlighting the cases where it is opportune to intervene in a more decisive manner. The evaluation of acceleration corresponding to reaching determined limits and the subsequent comparison with expected ground acceleration at a site allows on one hand, to judge whether or not the designed intervention is really effective (by comparing that current state and that of the project), and on the other hand, supply a measurement for the level of seismic safety of the building after the intervention.

As shown up till now, our focus is on the need of safeguarding buildings of artistic and historic importance. Structures exist, nevertheless, which hold strategic or relevant functions (i.e. hospitals, schools, military barracks, etc). In these cases, the priority for upgrading must be geared to the potential risk to the occupants and the operational functionality during a seismic emergency. As long as the safeguarding does not interfere with the operational strategic function or have other consequences in an eventual collapse, it is believed that over and above a certain level of risk, the possibility of delocalising the strategic and/or relevant functions should be taken into consideration.

The problem of finding a compromise between safety and conservation assumes important implications also in terms of the responsibility of the different subjects involved in the designing and execution of the work according to law. The owner must be aware of the index of the level seismic safety of the buildings after a designed upgrading intervention as well as its compatibility with the building's function.

Strengthening interventions may deal with single parts of the building or involve the entire structure. Thus two levels of evaluation have been introduced:

- LV2 – Evaluations to be adopted in the presence of local interventions on limited zones of the building which do not significantly alter the ascertained structural behaviour and for which local analysis methods have been suggested. In this case the evaluation of collapse acceleration of the whole, whenever requested must be effectuated with LV1 tools;
- LV3 – Intervention projects which modify the ascertained structural function or whenever an accurate evaluation is required for the seismic safety of a building. In this case, the evaluation must deal with the entire structure and local models may be utilised as outlined in LV2. However, they must be applied in a generalised manner on diverse elements of the construction, or on a global structural model when this is held to be necessary. The experience acquired following past seismic events has in fact demonstrated that for historic masonry buildings, in most cases the collapse was caused by the loss of equilibrium in limited portions of the construction (defined below as macro elements).

[4] It should be noted that no absolute value has been attributed to safety, thus allowing a compromise with conservation needs which is not different conceptually than that required by law for new buildings in which safety levels are defined as a compromise with construction costs.

## 2.3 Definition of reference limits for cultural patrimony

For architectural structures of historic interest, the safety and protection against seismic risk is guaranteed by defining two limit states which are motivated by the desire of safeguarding the occupants from danger in case of a rare and high intensity earthquake (ultimate limit state SLU), and to limit economical and functional damage in the event of more frequent low intensity quakes (limit state of damage SLD). Moreover, reasons for protecting specific works of art may exist (frescoes, stuccos, etc.) where it is advisable to establish a special limit state.

The limits states considered are:

- SLU (last limit state) – Under the effect of a reference seismic event which is characterised by a probability of exceeding 10% in 50 years and is defined below in point 3. It is opportunely modulated in terms of the differing probabilities of excess or the importance factor (see tab. 2.1). The structure even when submitted to grave damage, maintains a residual resistance and stiffness when confronted with lateral shifting and the entire load capacity when under vertical stress.
- SLD (limit state of damage) – Under the effect of a seismic event, characterised by a probability of exceeding limits by 50% in 50 years and defined below in point 3. It is opportunely modulated in terms of the differing probabilities of excess or a coefficient of importance (see tab. 2.2). The building on the whole is not greatly damaged in a way that justifies the interruption of use following an earthquake that has a major probability of occurring with respect to reference values for the last limit state.
- SLA (limit state of damage for works of art) – Works of art contained in a building (decorated walls, etc.) which during an earthquake of a certain level are submitted to modest damage and can be restored without significant loss of their cultural value.

With the SLU verification not only is the safety of the occupants guaranteed, but the preservation of the building which can be restored following an earthquake. The limit state of damage is considered only in relation to functional loss (declared habitable) of a building as far as damage to a historic masonry building is concerned especially with respect to frequent earthquakes, both of the intrinsic character of a building as the consequence of its acceptability as a whole.

In the case of the presence of important works of art in certain areas of the building, the evaluation shall be executed exclusively in those zones by utilising local models on parts of the autonomous structural elements (this kind of substructure is defined as a macro element; and can correspond to architectural elements, but more generally refers to constructive and historical recordings of the construction).

Generally, damage to protected works of art, i.e. frescoes and stuccos, becomes significant and not acceptable in the presence of grave damage to structural elements (cracking and deformities of bearing walls). These are used as a reference to decide whether the building may be declared habitable. In these cases of verification, an SLA can be compared to an SLD. However, situations exist where damage to decorated walls may be found even in the absence of structural damage (for example, stuccos of notable thickness which do not sufficiently adhere to a bearing wall) or vice versa (walls which are not totally bearing and are therefore able to hold up to cracking and structural deformation). In these cases, it is necessary to develop criteria and tools for specific SLA evaluations.

In conclusion:

- SLU evaluation is required for each protected building, even when not in use because it guarantees the safety of the occupants and the preservation of the building;
- SLD evaluation is required in the following situations:
  - On protected buildings as a whole when there is a desire to guarantee the functionality of a building in use following an earthquake;
  - Exclusively on a local level, in the parts of a building where protected works of art are located; in the presence of important works of art, the organisations which protect artistic assets may request a higher level of seismic protection to the limits that correspond to SLD levels in the event of an earthquake instead of the generally required SLU level.
- SLA evaluation is required in specific situations defined as Administrative Competency (for example, when the damage to decorated walls is found to be particularly grave even in the absence of structural damage).

## 2.4 Level of seismic protection

In paragraph 2.1 a safety index was introduced and defined as the ratio between acceleration that brings the work to a limit state and the expected acceleration of the site, which corresponds to a determined probability of exceeding the limit in 50 years[5]. The level of seismic protection of a determined work thus depends on the value of expected acceleration, and therefore of the probability of exceeding acceptable limits.

For architectural landmarks of historic and artistic interest, it is opportune to compare the index of seismic safety in current situations and those eventually attainable by strengthening interventions that are compatible with the needs of protecting the cultural heritage, with a level of seismic protection differentiated on the basis of their importance and use. Therefore the consequences may be ascertained more or less important in the event of seismic damage. With this goal in mind, the institution of the below in recommended:

- Three diverse "importance categories" (limited average, and elevated) which may be defined on the basis of the knowledge of the work by way of methods developed by the Ministry of Cultural Heritage and Activities (Attachment A, point A.15), by way of an interdisciplinary procedure;
- Three diverse "use categories" (infrequent or unused, frequent, very frequent); (Attachment A, point B.6).

According to its established category, each protected building shall be assigned a probability of exceeding acceptable limits.

For the verification of SLU, it is possible to refer to seismic events which are characterised by the probability of exceeding limits in 50 years as reported below in table 2.1 and differentiated from the limits set for new constructions. Such actions may also be obtained by multiplying the reference seismic action (probability of exceeding limits by 10% in 50 years) by a factor $\gamma_I$ called the importance factor, given the ratio between corresponding acceleration and the reference acceleration. In table 2.1 values of probability of exceeding limits and the average of $\gamma_I$ to the sixth power are shown which can be utilised whenever the corresponding acceleration of the differing acceptable probabilities is not directly available. The value of $\gamma_I$ and of the probability of acceptable exceeding limits is applied also in cases where the protected building is classified with an importance of I or II[6].

Table 2.1 – Probability of exceeding limits in 50 years of a seismic event (P) and factors of importance $\gamma_I$ for the verification of the SLU of protected buildings.

| **Category of use** | **Category of importance** | | | | | |
|---|---|---|---|---|---|---|
| | **Limited** | | **Average** | | **Elevated** | |
| | P. of excess | $\gamma_I$ | P. of excess | $\gamma_I$ | P. of excess | $\gamma_I$ |
| Infrequently or not used | 40% | 0.50 | 25% | 0.65 | 17% | 0.80 |
| Frequent | 25% | 0.65 | 17% | 0.80 | 10% | 1.00 |
| Very frequent | 17% | 0.80 | 10% | 1.00 | 6,5% | 1.20 |

In order to verify the SLD, one can refer to seismic actions characterised by the probability of exceeding limits in 50 years as shown in table 2.2, which can also be obtained by multiplying seismic actions corresponding to the probability of exceeding limits by 50% in 50 years by the average $\gamma_I$ factor as shown in the same table[7]. Whenever the acceleration value corresponding to the probability of exceeding limits by 50% in 50 years is not available, it can be obtained by dividing the reference acceleration $a_g$ by 2.5.

[5] The ground acceleration values in conditions of rigid sites (ground A) are furnished by seismic danger studies. In particular the reference acceleration $a_g$, of the NTC and of the Ordinance corresponds to a probability of exceeding limits by 10% in 50 years.

[6] Defined in point 4.7 of the Ordinance, Table 4.3. As a consequence of an eventual collapse, it is opportune to also keep in mind the induced risk of structures in an urban context. An example of this is the possibility that the fall of a bell tower obstructs an escape or emergency route is not substitutable.

[7] For the SLD, the values of $\gamma_I$ are obtained by taking an average of the ratio between the corresponding acceleration to the probability of exceeding limits as shown in table 2.2.andthe corresponding acceleration of the probability of exceeding limits by 50% in 50 years.

Table 2.2 – Probability of exceeding limits within 50 years of the seismic action (P) and importance factors $\gamma_I$ in order to verify the SLD of protected buildings

| **Category of use** | **Category of importance** | | | | | |
|---|---|---|---|---|---|---|
| | **Low** | | **Medium** | | **High** | |
| | prob. of exc. | $\gamma_I$ | prob. of exc. | $\gamma_I$ | prob. of exc. | $\gamma_I$ |
| Unused or rarely used | 90% | 0.50 | 80% | 0.65 | 65% | 0.80 |
| Frequent | 80% | 0.65 | 65% | 0.80 | 50% | 1.00 |
| Very frequent | 65% | 0.80 | 50% | 1.00 | 40% | 1.20 |

The index of seismic safety represents the ratio between the level of attainable seismic protection, which is compatible with the necessities of protected buildings and the desired level of protection in function of use and importance categories. This index is a reference value for the evaluation of the compatibility of the planned use, and as a consequence, for the operational decisions for necessary strengthening interventions and the general provisions to be adopted for guaranteeing safety (accessibility limits, limitations, or modifications of the end use, etc.).

## 2.5 Structural models, seismic analysis and design of strengthening interventions

For the evaluation of seismic safety and the project of strengthening interventions of protected cultural heritage, it is advisable to:

- Choose the reference seismic action for the site on the basis of the most advanced knowledge of seismic danger, avoiding the rigid way of subdividing into seismic zones. Such an action should take into consideration the foundation terrain categories and, if deemed necessary, can be further modified on the basis of seismic micro-zoning studies;
- Define a reference level of seismic protection, on the basis of importance and the conditions of use of the building;
- Obtain in-depth knowledge of the structure, which allows the identification of characteristics of the elements that determine structural behaviour. In the case of trials, even if slightly destructive, the impact of the same on the preservation of the building must be evaluated. This should be limited to those held to be truly necessary for the analysis. Based on the level of knowledge attained, an opportune confidence factor shall be assigned to establish the level of uncertainty of the model;
- Adopt one or two mechanical models of the structure or of its parts (macro elements) that are able to indicate the response during dynamic seismic actions, and coherently choose one or more analysis methods in order to conduct evaluations with a level of accuracy adequate to the objectives of the study. When possible, the model must be identified and validated on the basis of the behaviour manifested by way of the actual state of damage, especially if due to seismic phenomena;
- Express a positive judgement on the ratio between attained seismic safety by way of interventions that are compatible with needs of preservation, and the reference level of protection, desirably in relation to the seismic danger and the condition of use. Such judgement shall be expressed in global terms, not only on the basis of a numeric comparison between the collapse acceleration and the expected acceleration of the site, but also considering other aspects which were qualitatively evaluated, and may not be explicitly shown by calculations;
- Adopt adequate detailed rules in the realisation of strengthening interventions geared towards guaranteeing the compatibility of the new elements with the original ones including: durability of the materials, the maximum functional capability of the structural elements, and the construction as a whole.

# 3 SEISMIC ACTION

## 3.1 Identification of ground types

With the aim of evaluating seismic actions, the same categories used to define foundations for ordinary buildings may be adopted.
The classification is effectuated on the basis of values of the average shear wave velocity $V_{s,30}$, measured by depth H which includes the space between the floor placed on the foundation up to the roof of the rigid formation at the base having a propagation velocity of the shearing waves $V_s$>800 m/s (reference substratum). Whenever the roof of the reference substratum at the same depth as the floor placed on the foundation is found to be superior to 30 m, one assumes therefore that H = 30 m.
In the cases that the foundation is made of layers of large and fine grain earth falling within classes A and E, when direct measurements of the shear wave velocity are not available, one can nevertheless proceed as follows: a) Identify the layers of the two types of grain as a whole entering the total depth H, the parameter values of resistance $N_{SPT,30}$ and $c_{u,30}$ respectively for the layers of large grain earth and fine grain earth;
b) Identify the corresponding classes individually to the parameters $N_{SPT,30}$ and $c_{u,30}$;
c) Classify the foundation into the worst category among those identified in points a and b.
This approach is particularly useful in the cases where evaluation of seismic safety needs to be effectuated on a territorial scale (LV1), since such parameters are often already available from previous geognostic surveys at the site of the protected landmark or nearby.

## 3.2 Definition of horizontal ground acceleration

For the nation, the definition of seismic zones, which divides the national territory into areas characterised by differing levels of seismic hazard, is justifiable for its simplicity for protecting new structures, but may result to be too precautionary in many cases if it refers to the protection of existing buildings. Elevated values of lateral acceleration may induce projects for unjustifiably invasive strengthening interventions, especially in the cases of particularly important historic, architectural, and artistic landmarks.
It is more correct to define the reference acceleration $a_g$, as the peak lateral ground acceleration at the foundation of category A with a probability of exceeding limits by 10% in 50 years, until more detailed research can be conducted, this value can be utilised on specialised seismic hazard maps.
The reference values for the site can be obtained on the basis of available data of declared scientific value corresponding to the geographic coordinates of the building or to the public authority where the same is located.

## 3.3 Response spectrums

The reference model for describing ground velocity is constituted of an elastic response spectrum that is defined in relation to differing ground categories.
In the cases of low magnitude earthquakes and in general when verifying the limit state of damage, it is possible to adopt different spectrums for each ground type starting from the definition of the two types outlined in European Code 8 and the NTC.
Different spectrums can be adopted following the specialised analysis of local amplification of seismic motion according to the modalities indicated below.

## 3.4 Site amplification effects

Ground velocity is strongly influenced by the dynamic characteristics of the most superficial layers of terrain and the morphology of the site.
In fact, many times the effects of amplification of seismic motion were found both in terms of maximum acceleration as well as the contained frequency in the presence of particularly deforming flood deposits. The response spectrums defined for the differing categories of foundation terrain are an approximate tool to

measure some of the aspects cited above. In particular, the influence of the most superficial layers and the most deformable terrains and in the presence of impedance contrasts.
Amplification or deamplification of motion was encountered in the presence of diverse morphological configurations: crests, ridges, slopes, and subsidence. Morphological effects can be considered to increase seismic action through a coefficient of topographical amplification or on the basis of studying local seismic response.
In some cases, it may be advisable to analyse the onsite effects in a more complete way through surveys of seismic micro-zoning. In this case it would be possible to consider the following factors: the presence of active, genetic, seismic fault lines, the possibility of activation by permanent shifting caused by mudslides, liquefaction and/or densification.
Whenever more accurate determinations of local seismic motion are performed by way of micro-zoning surveys, specific spectrums for the site should be utilised.

# 4 KNOWLEDGE OF THE BUILDING

## 4.1 The path to knowledge

### *4.1.1 General description*

Knowledge regarding historic masonry buildings is presumed to be fundamental as a means of both reliably evaluating current seismic safety as well as the choice of effective strengthening interventions. The common problems for all existing buildings even in the case of protected ones, given their importance, is the impossibility of knowing the original characterising data of the structure, the modifications sustained over time due to damage derived from phenomena of anthropic transformations, the aging of materials, and natural calamities. Moreover, the execution of a complete research campaign may prove to be too invasive on the fabric of the building itself.
Thus, the necessity of refining analysis and interpretation techniques of historic buildings must use means of cognitive phases of diverse gradations of reliability that also relate to their eventual impact. This knowledge indeed may be attained at various levels of depth depending on the accuracy of the surveying operations, historic research, and experimental studies. These operations will be the result of proposed objectives and will interest either the entire building or a part of it according to the typology of the proposed strengthening intervention. The study of the characteristics of the construction relies on the definition of an interpretive model that allows, during the various calibration phases, both a qualitative interpretation of the structural function as well as the structural analysis for a quantitative evaluation. The grade of reliability of the model is tightly linked to the level of research and the available data. From this point of view, different levels of knowledge are introduced at increasing levels of depth to which confidence factors can be linked that can be utilised in the final analysis both for the actual state and after eventual interventions.
The path to knowledge may be broken down into the following activities:

- The identification of the building, its location in relation to particular risk areas, and the rapport of the same within its surrounding urban context; the analysis consists in an initial schematic survey of the building and in the identification of eventual noteworthy elements (decorated fixed to the walls, antique furniture) that may condition the level of risk;
- The geometric relief of the building in its actual state, intended as the complete stereo metric description of the structure including eventual cracking and deforming phenomena;
- The identification of the evolution of the building, intended as the sequence of the phases of constructive transformation, from the hypothetical original configuration to the present state;
- The identification of the elements which make up the resistance organisms, in the material and constructive acceptance with particular attention turned to construction techniques to construction details and their connections to the other elements;
- The identification of material, their state of decay and their mechanical properties;
- The knowledge of the foundation and its structures with reference to variations that occurred over time and relative instability as well.

In consideration of the specific modalities of the structural analysis of collapse mechanisms in historic masonry buildings as described in chapter 5, the cognitive research must be focused primarily on identifying the history of the building, the geometry of its structural elements, construction techniques, and phenomena of instability and decay.
On the other hand, difficulties connected to knowledge also in relation to available resources and the invasiveness of the research to be performed, often makes it necessary to compare by means of interpretive and posterior models based on the observations and verification of the shown functioning of the building.
The acquired information must be organised and returned according to the monitoring program of the state of preservation of protected architectural landmarks as outlined in Attachment A, and elaborated by means of forms from the Ministry of Cultural Heritage and Activities and finalised for the acquisition of systematic knowledge of Italy's Cultural Patrimony.
The following paragraphs describe in detail the diverse phases of knowledge; it should be noted that they are not to be intended as sequential, but integral.

#### *4.1.2 Identifying structures*

The first step to knowledge consists in the correct and complete identification of the organism and its localisation in the territory as an ends of identifying the sensibility of the structure with respect to different risks and in particular, seismic risk. This phase of the analysis includes an initial schematic survey of the building in the form of a simple sketch, able to describe the maximum consistence and locate eventual elements of valour whose importance and condition of risk may be determined. The identification of elements of valour must be utilised also for identifying possible zones of sacrifice where eventual destructive surveys may be realised and the placement of eventual strengthening interventions.
In this phase, the rapport between the building and its surroundings must be analysed by way of a description of the Architectonic Complex (CA), isolated and non-isolated, and the characterisation of the spatial and functional rapport between the building and eventual adjacent buildings. The study of the fabric must permit the hypothesis of the constructive hierarchy and the relation between the building and its context. In particular, beginning from the concept of Architectonic Complex (CA) consisting in the aggregation of more structural units and spatially defined from the streets that surround it, one must arrive at the Composed Fabric units (CF), identifiable by the analysis of the visible prospects and the in-plane altimetry junctions. The survey may be effectively conducted by way of macro stratigraphic techniques.
The results of this knowledge phase may be reported by utilising forms A and B, as described in Attachment A.

#### *4.1.3 Functional characteristics of the building*

Knowledge of the work cannot exclude analysis, including historic, of the functioning of the building and its junctures, finalised as to recognise their uses and their succession over time and in what environment. The result of this analysis leads to the availability of useful information for understanding the motivations behind structural and geometric modifications which happened over time, for motivating eventual signals or noted instability, for planning possible future utilisation which is compatible with the characteristics of the building, and with the objective of reducing seismic risk.

#### *4.1.4 Geometric surveys*

The knowledge of structural geometry of existing masonry organisms is derived from survey operations. Surveys must refer to both the complete geometry of the whole as well as the construction elements, including their rapport with any eventual adjacent buildings.
The stereo metric description of the structure permits the identification of the characteristics of the construction elements, both in plan and in elevation. Consequently, at each level, the geometry will be surveyed for all of the masonry elements, the vaulted arches (thickness and profile), of ceilings and roof coverings (type and warping), of the stairs (structural typology), the localisation of eventual niches, cavities, blocked openings (with what modality), chimneys, any included extraneous elements, and the typology of the foundation.

The representation of the survey will be effectuated by way of plans, elevations, and sections besides detailed construction particularities. Once this operation has been completed, one shall proceed with the identification of the resistant structural scheme inside the total geometric survey.
Because the geometric relief serves to define the geometry of the model to be utilised in the seismic analysis, its boundaries and load agents, it is advisable to verify that all of the necessary information has been found. In particular, the most significant points should be identified for the calculations model, where horizontal measures and vaulted arch systems are set as well as the support mechanisms of the nearby walls. Moreover, the mass and the loads of each wall element must be completely determinable.
The description of the rapport between the elements can utilise recording techniques and restitution of their own stratigraphic analysis. The procedure can be articulated in virtue of the typological characteristics of the structure and the territorial and urban context it belongs to.
The difficulties of geometric surveys are linked to the accessibility of certain spaces, i.e. attics, volumes hidden in fake arches or dropped ceilings and roof coverings. Another difficulty is found in the excessive height of the elements to be measured such as bell towers, towers, vaulted arches in a nave, etc. All in all, tools are available for complex elements too, and direct surveying techniques (Endoscopy) or indirect ones (thermography, geo-radar, etc) may be used for inaccessible spaces. The three-dimensional rendering of the organism can be rather complex, but it is certainly useful for creating the model. Stucco relief, decorations, and non-structural elements while useful for identifying the work of art and its importance, must not hinder the detailed knowledge of the geometry of the architectural and structural elements.
Any eventual cracking must be shown and represented as to allow the identification of the causes and possible evolution of the structural problematic of the organism. The lesions shall be classified according to their geometry (extension and width) and their kinetic mechanisms (detachment, rotation, sliding, out-of-plane shifting). Subsequently, considering that the phases of knowledge are not sequential one or more damage mechanisms can be associated to each lesion or group of lesions, which are compatible with the geometry of the body and its foundations, with the transformations sustained, with the present materials, and the events sustained. In a similar manner, the deformities should be classified according to their nature (evidently out of plumb, sinking, swelling, bulging walls, fallen vaulted arches, etc.) and when possible associate each to their respective damage mechanisms.
The geometric relief must be integrated with the compilation of graphic scheme forms called morphologies (form C in Attachment A), which allows the univocal identification of resistant elements and their relative constructive rapport.

### *4.1.5 Historical analysis of earthquakes and of subsequent interventions on the building*

As a way of correctly identifying the structural system and the stress state of each element, it is important to reconstruct the entire constructive history of the protected building, or in other words, the construction process and all later modifications over its life. In particular, the sequence of the various portions of the structure should be shown with the aim of identifying the areas of possible discontinuity and lack of homogeneity in materials both in the floor plans as well as raised structures (additions, height additions, substitution of ceilings, etc.).
The history of the building can also be utilised as a tool for controlling and verifying the response of the building and particular natural or man-made events and their subsequent transformations. The events sustained by the building must however be identified especially the most significant and traumatic ones, and the corresponding effects, ascertained by proper documentation (written and iconographical reports) or through a direct analytical relief of the building. The history of the building's use may also furnish indications of the actions applied in the past.
The knowledge of the response of the construction to a particular traumatic event may permit the identification of a qualitative behavioural model, even if alterations to the construction over time must be taken into consideration, especially immediately following that event. This analysis shall be the guide for the definition of the most critical damage mechanisms and for the subsequent definition of reliable calculation models.
As a means of understanding the current behaviour and for defining eventual seismic strengthening interventions, it is important to identify the nature of the strengthening interventions already realised in the past, the localisation, and the structural elements involved, the period of their realisation and the verification of their effectiveness over time.

The consultation of numerous existing seismic catalogues[8] and the direct gathering of archival documents relative to the seismicity of places and the damage sustained of the buildings make up a fundamental reference basis. The analysis of catalogues of sites allows in cases where one can assume that the data is statistically complete, to compare the actions assumed as a reference for the differing limit states, deduced from the seismic danger maps with the seismicity of the site. In certain cases, it may be advisable to keep the seismic history of the site in mind in the final judgement of the evaluation of seismic safety, considering the intensity experienced in the past contains the local effects within it.

#### *4.1.6 A survey of construction materials and conservation states*

The constructive material survey must permit the complete identification of resistance mechanisms of the building keeping in mend the quality and the state of preservation of the materials and the construction elements.
Such acknowledgement requires the acquisition of information which is often hidden (under plaster, hidden behind dropped ceilings, etc.), which can be performed thanks to non-invasive, indirect surveying techniques (thermography, geo-radar, ultrasound tomography, etc.) or the slightly invasive direct inspection (Endoscopy, scraping away layers of plaster, samples, small breaking, etc.). An important aspect is the choice of number, of the typology and the localisation of the trials to be performed. For a correct knowledge, they should be adopted diffusely, but due to their eventual impact and for economic reasons, they should be employed only if well-motivated, or in other words, when they are useful for the evaluation of the intervention project. As an ends to limiting the impact of these surveys as much as possible, beyond the knowledge of the constructive story of the building in question, it is fundamental to have an in-depth understanding of the constructive characteristics of the buildings in the area and in the diverse historic periods as a means of falling back on characteristics deduced of the *regola dell'arte* (well-done work).
Special attention should be reserved in the evaluation of the quality of the masonry including the geometric and material characteristics of each component, as well as the modality of assembly. The following elements proved to be particularly important:

- The presence of transverse elements (stones or bricks) which connect the wall leaves; the shape, type and size of the elements;
- The acknowledgement of the regular placement and practically horizontal courses (or, alternatively, the presence of regularly stepped bordering);
- The good composition, obtained by way of the mesh of the elements (number and range of contacts, presence of scales) and the regular staggering of the joints;
- The nature of the lime mortar and its state of preservation.

Considering the notable variety of materials and techniques, both on a geographic and historic level, it is useful to define local *rules of thumb* for the quality judgement reference of a wall.
The recording of a scheme of structural functionality of the building necessitates knowledge of constructive details and the characteristic of unions between the various elements:

- Typology of walls (in brick, squared-off stone, rough-hewn stone, split, pebbly or mixed, unique parameter, or with two or more parameters) and constructive characteristics (regular or irregular texture; with or without transverse joints, etc.);

[8] List of the main seismic catalogues for the Italian territory:

- Postpischl D. (1985), Catalogue of Italian earthquakes from the year 1000 to 1980, Financed Geodynamic Project (PFG) of the C.N.R.
- Camassi R. and Stucchi M. (1997), NT4.1 – A parametrical catalogue of earthquakes in the Italian territory which surpassed the damage threshold, National Group for the Defence against Earthquakes (GNDT), http://emidius.mi.ingv.it/NT/.
- Monachesi G. and Stucchi M. (1997), DOM4.1 – a macro seismic observational database of earthquakes in the Italian territory which surpassed the damage threshold, GNDT, Milan-Macerata, http://emidius.mi.ingv.it/DOM/home.html
- Boschi E. and company (1997), CFTI – Catalogue of high intensity earthquakes in Italy (Version 2 - from 461 AD to 1990), National Geophysics Institute (ING) / SGA Environmental Geophysical History (Bologna), http://storing.ingv.it/cft/.
- National Geophysics Institute (ING) / National Group for the Defence against Earthquakes (GNDT) / Environmental Geophysical History (SGA) / National Seismic Service (SSN), CPTI – Parametrical catalogue of Italian Earthquakes, 1999 Edition
- Work Group for the mapping out of seismic danger charts (PCM Ordinance 3274/03), Catalogue of earthquakes CPTI2, National Institute of Geophysics and Volcanology (INGV), 2004.

- The quality of the unions between vertical walls (clamping in the corners and in the hammers, tie rods, etc.);
- The quality of the lateral joints (ceilings, arches and roof coverings) and walls, with surveys of the eventual presence of in-plane stringcourses or other connecting systems (tie rods, etc.);
- Elements of discontinuity determined by cables, chimneys, etc.
- Typology of horizontal structures (ceilings, arches, roof coverings) with particular reference to their in-plane stiffness;
- Typology and effectiveness of the architraves above openings;
- The presence of structurally efficient elements chosen to balance any eventual thrusts present;
- The presence of highly vulnerable but not necessarily structural elements.

The results of the constructive material survey may be articulated by way of the realisation of graphic schematic forms (form D in Attachment A), chosen to describe the individual elements and their relative state of preservation. The graphic schematic form shall be enriched by the information about the rapport between the elements and in the case of urban aggregates, of their relationship with adjacent buildings.

#### *4.1.7 Mechanical properties of materials*

The visual survey of certain research may allow the addition of good knowledge and to a judgement regarding the quality of materials and their decay (point 4.1.5). All in all, in some cases the modelling of structural behaviour especially with respect to seismic activity requires the knowledge of mechanical parameters about deformity and resistance of the materials, and particularly, the masonry.
Non-invasive, indirect diagnostic techniques, i.e. sonar and ultrasound, allow the evaluation of the homogeneity of the mechanical parameters in the diverse parts of the construction, but don not furnish reliable quantitative estimates of the values due to deductions regarding size and other measurements (i.e. the propagation sonic waves).
Direct measurement of mechanical parameters of the masonry especially resistant ones cannot be performed however without trials which are slightly destructive or destructive even if on limited portions. The combination of non-invasive trials with more invasive ones may be utilised for reducing the invasiveness of the qualification surveys.
The characterisation of the components (mortar, bricks or stones) can be performed on site or on small-sized samples taken from the site and later examined in a laboratory. As far as the mortar is concerned, it can be tested in these methods and others in order to characterise it: a) sclerometric and penetrometric trials; b) chemical analysis, on samples taken from inside the wall so as to not have been effected by surface decay. On the bricks, besides determining the physical characteristics, it is possible to evaluate a plastic model and resistance under traction and compression by way of mechanical laboratory trials of compression and flexion. As far as the stone elements are concerned, lithological tests can be performed.
The mechanical characteristics of masonry can be deduced from the properties of the elements they are composed of only when dealing with brick masonry or natural regular textured rough-hewn stone. In these cases, it is possible to refer to the indications contained in paragraph 5.4 of the NTC or in other documents of recognised scientific and technical value.
In other cases, it is possible to utilise the following on-site trial modalities:

- For determining the normal plastic model and resistance under compression: a) double flat hammer (slightly destructive technique because performed on a limited portion of a parameter wall which has sustained a maximum solicitation corresponding to the trigger of cracking. This should be realised by means of a small cut preferably performed inside the mortar junctions, and therefore easily restorable; b) compression trials on a wall panel (very invasive due to the involvement of an important section of the wall, the order of the meters, and requires the execution of a rather large cut in order to place hammers and often ponderous contrast structures). The trial b) should be limited to cases where other investigative methodologies do not supply sufficiently reliable evaluations or when walls that can be sacrificed are available;
- For determining resistance and the shear model, generally trials on panels can be utilised because of the considerations stated above according to two modalities: a) diagonal compression trials on a square panel; b) Compression and shearing trials on a rectangular panel where the height is twice the width. Both of these trials are of a highly invasive nature.

It is evident given the destructive nature of the trials sustained, that they should be employed only if well-motivated and justifiable not only in light of the relative results in the structural modelling, but also by the

fact of being discriminated against compared to evaluations or intervention choices. In the identification of possible sacrificial zones where eventual destructive analysis can be carried out, the results of historic research, the state of preservation of the materials and the relief of valuable surfaces should be taken into consideration. The number of trials that can be carried out on homogeneous material will usually be very limited and will not allow a statistical treatment of the significant results in relation to formal verification procedures regarding safety based on probabilistic or semi-probabilistic methods. Planning the surveys and interpreting their results should form part of a more complex procedure in which even only one piece of experimental data can take on a significant role.
The identification of mechanical characteristics may also be obtained by comparison with similar walls, whenever possible, while keeping in mind decay phenomena. With this aim, it is desirable that territorial protection and control entities institute permanent archives which contain: a) almanacs of different masonry walls present throughout the history of the area in question; b) tables with reference values for mechanical properties, deduced from experimentation organised by the aforementioned entities and/or utilising campaigns done for individual interventions and studies.
The data acquired in the research campaigns should be returned in such a way that allows, in a relatively short period of time, the creation of a databank that would be accessible by way of a monitoring program of the state of preservation of protected cultural heritage (Attachment A).
In the absence of the aforesaid archives, for each type of masonry the average values of mechanical parameters should be referred to as defined in Attachment 11.D of the Ordinance.

#### *4.1.8 Terrain and foundations*

Knowledge of the foundation and the existing foundation structures are of primary importance for the forecasts of seismic behaviour. In the case of protected cultural heritage, such knowledge must not be limited to modifications sustained over time due to natural or man-made causes, i.e.: excavations, adjacent construction, variation in the drainage scheme, hydrogeological shifting.
For correct knowledge of terrain and foundations, the tests listed below should be adopted diffusely, but for their eventual impact and economic reasons, they will be employed only when well-motivated. In other words, they should be performed whenever it is justifiable not only because of the use of the relative results of the foundation and terrain modelling, but also due to bias in the evaluation or in the choice of strengthening interventions.
An accurate geotechnical characterisation of the foundation has the following objectives:

- The definition of a geotechnical model of the foundation on the basis of surveys performed in correspondence to a sufficient number of boreholes to elaborate profiles and stratigraphic reference sections for the analysis. In order to do this, surveys that utilise continuous logging must be performed along with taking undisturbed representative samples of the involved terrain. Penetrometric trials have proven to be of great usefulness for the stratigraphic characterisation, which may be performed with the integration of surveys to reduce the number of samples needed.
- The definition of the underground waterways and their seasonal variations, including eventual incidences of man-made interventions such as various draining techniques already in act or planned in the future. This goal should be followed by way of the installation and monitoring of well-positioned piezometers. For work that interests extended portions of the territory, the research should be supported by hydrogeological hydro-geological studies.
- The mechanical characteristics of the various deposits that make up the foundation necessitate the definition of parameters for the analysis in order to quantify the seismic risk which the building being studied is exposed to as well as the surrounding terrain. The analysis is aimed to show:
  - The dynamic interaction of terrain, foundation, and structure to verify the possibility of structural damage or the collapse of the foundation after a seismic event. To this end, the knowledge of the following is indispensable:
    - (i) Of the shear resistance in draining and non-draining conditions in the presence of cyclical solicitation:
    - (ii) Of shear deformity modules and the damping coefficient, as well as their dependence on the initial state of deposit and of the level of tangential deformation. This knowledge may be obtained from laboratory experiments on undisturbed samples, with underground or surface geophysical trials (i.e. determining the velocity of shear waves

with *cross hole, down hole, SH* and *MASW wave refraction* techniques*)*, with a wide range of on site trials already mentioned in the section on penetrometric testing;

- The susceptibility of the foundation to liquefying and the cyclic mobility in occasions of seismic activity/events. The verification of liquefying or cyclic mobility, when deemed necessary in relation to the characteristics of the terrain, is led generally by referring to peak lateral ground acceleration with horizontal in-plane campaigns and in the absence of constructions. It is also necessary to follow simplified methods based on the results of SPT or CPT trials or on the measurements of the shear wave velocity. In the cases where risk of liquefying of excessive shifting due to cyclic mobility is verified, specific strengthening interventions and terrain reinforcements can be considered;
- The verification of the stability of natural slopes in the vicinity of excavations and support operations with respect to collapse or excessive shifting which may compromise the stability of the structure in question or compromise its functionality. The geotechnical parameters needed to verify have already been mentioned for matters regarding the dynamic interaction between terrain and structure. In the case of natural slopes that had already sustained mudslides in the past, the mechanical characteristics of the terrain must include the determination of the remaining resistance. Whenever these verifications are not satisfactory to pass habitability or safety requirements, consolidation measures of the terrain and/or modifications to reinforcement intervention may be enacted. Among the numerous techniques of reinforcement, consolidation and drainage available from geotechnical engineers the best for each situation should be chosen.

Besides the geotechnical characteristics of the foundations, it is necessary to conduct research that allows the definition of the geometry and characteristics of the existing foundation, and to ascertain the eventual state of deterioration and ruin. The first step in this direction is the gathering of all available news about the original foundation and any eventual modification it sustained in the past. Particular attention whether there is an archaeological substratum present for the fundamental role that such a substratum may have in altering the seismic response of the entire structure and in limiting the possible types of interventions on the foundation and terrain.

When this information has been obtained, a survey knowledge program can be planned and actuated based on:

- Wells or ditches, excavations along the perimeter of the building starting from the basement or the exterior, and pushed till they reach the floor where the foundation was posed. It must be ascertained whether these excavations had a negative effect on the static behaviour of the building.
- Perforations with continuous logging, oriented in various directions through the walls of the foundation until they make contact with the earth below. The techniques and the utensils for perforations must minimize the disturbance to the walls and optimise the quality of the logging devices in order to prepare them for laboratory testing. The logging work on foundations is invasive diagnostic instruments, and therefore must be used only when strictly necessary. The holes of the logging may be utilised for later inspections with a telescopic probe, in order to conduct geophysical tests and if requested absorption tests.
- Geophysical methods, which often represent an effective and non-invasive diagnostic tool. Georadar and sonar, thermal, and electric tomography are particularly effective for evaluating the mechanical properties of masonry and its state of decay. These techniques require highly trained experts to utilise the equipment, systems of data input and adequate elaboration software, which must clearly be documented.

The survey results allow the verification of the vulnerability of the foundation and bring to light the necessary seismic improvement, which must be harmonious with the designed interventions on the raised structure.

#### *4.1.9 Monitoring*

The periodic control of the construction is strongly advisable because it represents the primary tool for knowledge regarding preservation. It allows the planning of maintenance operations and the timely enacting of reparation interventions in case of structural damage, and consolidation for the sake of prevention only when it is really necessary.

To set up a monitoring program, it is first necessary to perform an accurate analysis of the structural function, then interpret the deterioration present so as to define the most significant parameters that when measured regularly and within an adequate timeframe allow the guarantee of good behaviour or evaluate eventual dangerous evolution of the stability of the whole or single parts of the building.
Visual monitoring intended as the periodic control of the onset of cracking, phenomena of decay, transformations in the structure and in the surrounding environment represent a starting point of such an activity.
Additional information may be acquired through instrumental monitoring of certain parameters, which are retained significant (movement of the lesions, absolute shifting or relative to construction points, rotation of walls or other elements). The movement of lesions may be controlled continuously and by radio control. However, it is necessary to consider that in relation to the kind of deterioration, the danger thresholds of this shifting may vary greatly from one example to another. The geometric control of the construction can be performed by means of a procedure of topographical relief, photographically, or by utilising innovative techniques such as a mass of points generated by a *laser scanner* (obviously this method is not exclusive and its relation to the motion threshold retained significant must be carefully and precisely evaluated). The monitoring project requires a preliminary interpretation of the mechanism of deterioration. This can often be done thanks to balance mechanisms of the masonry considered as a stiff/rigid body. Then a numerous series of points to control can be identified. In certain cases, when the eventual deterioration is well understood and the threshold of safety can be defined, monitoring represents an alternative to intervention to the advantage of preservation.
The dynamic properties of the structure (frequencies and the singular vibratory forms) are also used as significant parameters of the behaviour of a construction. In the presence of deterioration or transformation of the building and its end use, these measures undergo alterations. At the actual state of knowledge, the identification of damage on the basis of dynamic properties is rather difficult. Moreover, it is well-noted that masonry buildings are characterised by behaviour which is strongly nonlinear which renders setting standard parameters of the equivalent linear behaviour problematic. Overall, the control of certain parameters of dynamic response, whether artificially caused or through environmental vibrations, in certain cases may represent one of the possible elements for identifying manifested changes in the construction. The choice of parameters and interpreting dynamic measures are justifiable in relation to the typology of deterioration and the aims of the survey.
If seismic safety is considered in further detail, seeing how earthquakes are rare and unpredictable events, it is evident that monitoring does not represent an advance alarm tool nor help to identify seismic behaviour. Its use may be significant in post-quake emergencies for highly damaged structures in the case where there is a desire to verify the evolution of the mechanisms activated by the seism and the response of eventual aftershocks.

### 4.2 Levels of knowledge and confidence factors

Once the constructions have been identified in relation to the depth of the geometric survey, the material and constructive surveys, and the mechanical research regarding the terrain and foundation, the designer assigns a confidence factor $F_C$, ranging from 1 to 1.35, which grades the reliability of the structural analysis model and the evaluation of the seismic safety index.
The confidence factor is applied differently depending on the model for evaluating seismic safety. As illustrated in chapter 5, they can be classified as follows:

- Models that consider the deformability and the resistance of the materials and structural elements;
- Models that consider the balance limits of the various elements of the structure, taking into account the masonry materials as stiff and non-resistant to traction (the creation of kinematic models of rigid blocks, by means of the introduction of justifiable separations).

In the first case, the confidence factor is applied to the properties of the materials, reducing both the plastic model as well as the resistance. The starting values of the mechanical characteristics to which the confidence factor is applied will be defined in intervals after the usual construction techniques of the age, on the basis of the resultants of the materials surveys and the construction details (for masonry, the intervals reported in Table 11.D.1 integrated with Table 11.D.2 of the Ordinance may be utilised).
In the second case, regarding rigid blocks, in which the resistance of the materials is not taken into consideration, the confidence factor is applied directly to the structural capacity. In other words, by

reducing the corresponding acceleration to the various limit states. Whenever surveys are conducted on the mechanical properties of masonry, a partial confidence factor $F_{C3,}$ a value lower than 0.12 may be assigned only when the resistance of the masonry under compression id considered in the evaluation model.
In both cases, the definition of confidence factors should refer to the material/typology, which most significantly penalises the specific mechanism of damage/collapse under question.
For example, the confidence factor may be determined by defining diverse partial confidence factors, $F_{Ck}$ (k=1,4), on the basis of the numerical coefficients reported in Table 4.1, in which values are associated to four categories of research and to the level of depth reached:

$$F_C = 1 + \sum_{k=1}^{4} F_{Ck} \tag{4.1}$$

Table 4.1 – The definition of the level of the depth of research on diverse aspects of knowledge and relative partial confidence factors.

| **Geometric Survey** | **Material and Construction Survey** | **Mechanical Properties of the Materials** | **Terrain and Foundations** |
|---|---|---|---|
| Geometric survey has been completed<br>$F_{C1} = 0.05$ | limited survey of materials and constructive elements<br>$F_{C2} = 0.12$ | mechanical parameters deduced from available data<br>$F_{C3} = 0.12$ | limited survey of terrain and foundations, in absence of Geological data or availability of information about the foundation<br>$F_{C4} = 0.06$ |
| Geometric survey has been completed, together with the graphic rendering of cracking and deformities<br>$F_{C1} = 0$ | extensive survey of materials and constructive elements<br>$F_{C2} = 0.06$ | limited research of mechanical parameters of materials<br>$F_{C3} = 0.06$ | Geological data and information regarding the foundation structures is available; limited research on terrain and foundation<br>$F_{C4} = 0.03$ |
| | exhaustive survey of materials and constructive elements<br>$F_{C2} = 0$ | extensive research of mechanical parameters of materials<br>$F_{C3} = 0$ | extensive or exhaustive research on the terrain and foundation<br>$F_{C4} = 0$ |

The geometric survey in any case, must be developed to a level of detail coherent with the needs of the adopted geometric model in the analytical evaluation and/or the necessary qualitative considerations.
The material research (typology and texture of the masonry, and the state of the floors, structures, and filled arches, etc.) and of the constructive details (wall clamping, eventual weaknesses, entities and types of horizontal supports, measure taken for thrust containment, material decay, etc.) must tend to be compatible with the necessities of protecting the building, ascertaining the various construction typologies present, their placement and repetition, with particular attention to all aspects which may trigger local collapse mechanisms.
When diverse structural materials are present, the level of depth and the consequent confidence factor $F_{C3}$ may refer to materials or the most significant materials that influence the assigning of the seismic safety index. When the seismic analysis is based on an evaluation related to a single local mechanisms (see 5.2.1, 5.2.2, and 5.2.4), partial confidence factors relative to each modelled portion may be utilised.
IN the case of evaluation of local characteristics when information regarding terrain and the foundations does not bear any relation to the specific collapse mechanisms, a partial confidence factor $F_{C4}$ of 0 may be assigned. In other cases, as far as knowledge regarding terrain and foundations is concerned, aspects linked to the definition of the ground categories should be distinguished from those relating to the transmittance of

actions of the structure to the ground (Geometry of foundations and geotechnical parameters of foundational terrain)[9].

## 5 STRUCTURAL MODELS FOR THE EVALUATION OF SEISMIC SAFETY

### 5.1 The seismic behaviour of historic masonry buildings

Historic masonry buildings are a combination of many extreme variables and complexities for their typology and construction techniques. So the analysis of their structural behaviour and the evaluation of their safety are conditioned by notable uncertainties in defining the mechanical properties of the materials and the conditions that connect the various elements.

These constructions were not designed utilising the principals of the material nor structural mechanics, but by an approach based on intuition and observation, utilising the principals of balance among rigid bodies and experimenting behavioural construction of existing building. All this, with time brought about the refinement of performance criteria and proportional geometry, also known as *rules of thumb*. Although this approach is not scientifically rigorous, it resulted to be reliable only if utilised within valid limits of the rule (as demonstrated by the experimental collapse in the past when those limits were exceeded). Acknowledging a building that conforms to *rules of thumb* standards can be the first element of safety evaluation. *Rules of thumb* may be acknowledged also by construction techniques, which show a local specificity the fruit of progressive refinement in the use of local materials available in that area (the variety of masonry types in relation to the characteristics of the components is emblematic of this).

Besides conforming to *rules of thumb*, a further evaluation element can be the "inspection" of history, to which the same construction bears. This inspection moreover, has often proven to be insufficient with respect to prevention in case of seismic risk, because the construction (albeit ancient) may not have ever been hit by a violent earthquake, which is normally assumed when evaluating safety with respect to the last limit state. An earthquake with a probability of exceeding limits by 10% in 50 years corresponds to a period of response of 475 years. In other words, the time that must pass between one earthquake and another of said intensity. However, the possibility exists that during the seismic history of a site, such an event may have occurred much earlier in time. Moreover, one must consider that the capacity of the structure may have been modified as an effect of accumulative damage due to past low intensity earthquakes, or shifting due to other causes and for transformations that often interest historic buildings.

It is wise to note that besides factors of an economic nature, the anti-seismic constructive culture in a geographic area has been influenced by its level of seismic hazard and the occurrence of earthquakes. In areas of high seismicity (characterised by the possibility of violent earthquakes and by the presence of significant earthquakes of a certain frequency), experience has provided effective construction solutions for the reduction of vulnerability such as: buttresses, tie rods, clamping, etc. These elements of an anti-seismic nature have become an integrative part of construction regulations, specific to each region with frequent earthquakes. In areas of low seismicity (rare and non-destructive earthquakes), instead such measures were put in act only during reparations or in restoration following the rare significant earthquake, but their use as part of construction was lost after a few generations, because they soon appeared unnecessary motivated.

Nevertheless, the evaluation of the seismic safety of an existing building is now evident. The aforementioned elements of evaluation must be accurately acquired by adequate knowledge, but in any case it is wrong to not perform a structural analysis, finalised to translate the ascertained behaviour of a building into mechanical and quantitative terms. To perform such an analysis various models are available which have proven to be more or less accurate and include either the entire construction or individual parts.

---

[9] For the first, in absence of relative information, the research must tend to exclude the presence of S1-S2 ground types, and is retained decisive in terms of planning choices and/or functionality (eventual changes in end use), to assign a macro-category (A, B-C-E, D).

For the second, the research may generally be omitted whenever the responsible and motivated judgement of a technical expert or when the following circumstances are verified regarding terrain characteristics: (a) No deterioration is present due to the sinking of the foundation; (b) interventions have not caused alterations in the structural scheme of the substantial construction for the transmission of solicitation to the terrain, nor have modifications been revealed in weight or overloading; (c) modifications are not underway for hydrogeological repair in the area which could influence the stability of the foundation. Otherwise, the surveys shall be limited to the evaluation of parameters that influence the unverified circumstances.

In Attachment B, the mechanical functions of masonry, the interaction between the various elements that form the whole historic construction and the possible strategies for structural modelling are illustrated.

## 5.2 Seismic analysis methods

### *5.2.1 Introduction*

In the case of existing masonry buildings, it is possible to fall back on diverse analysis methods, depending on the model which best describes the structure and its seismic behaviour.
In the case of cultural heritage, the evaluation of the structural capacity and its seismic safety will be effectuated, on both a local level as well as on the whole, utilising the most adept analysis method. In particular, it is possible to refer to the following:

- Static linear analysis
- Dynamic modal analysis
- Nonlinear static analysis
- Nonlinear dynamic analysis

Below, the conditions and limits for utilising the aforementioned analysis methods are illustrated relating specifically to historic buildings.

### *5.2.2 Static linear analysis*

Reference seismic actions at ground level, for the ultimate limit state, are reduced in this case through the structural factor to allow the verification in an elastic field. In this way, an implicit account of further capacity for displacement is made, when the resistance limit is reached, before the structure arrives at the ultimate limit state. It should be noted that applying this method to historic buildings may be problematic due to difficulties in defining appropriate structural factors as well as possible consequences in defining interventions.
In the case of elastic linear analysis on finished elements, the model must undergo a lateral-force system whose nature is defined in the Ordinance (point 4.5.2). Such forces may be distributed by reason of the measurements of the various masses, according to the Ordinance indications, only when the buildings are similar from a structural point of view to an ordinary building. In other cases, the following must be assumed: a) a distribution of force proportional to own masses; b) a distribution of force proportional to the main vibration modes in the direction of the analysis, estimated on the basis of the distribution of stiffness and of the masses of the various elements and eventually corrected with iterative procedures. The period of vibration may be estimated with the formula indicated in the Ordinance only when the structures are comparable. For all other structures, the wisest choice of formula must be selected or invented starting from the main modal form, adopting, for the materials, the values of cracked elastic models.
The assumed structural factors must be justified by the displacement capacity of the structure in areas where cracking/fracture is found, evaluated on the basis of both the typology of the building as well as the construction quality (materials, construction details, junctions). Accidental torsional effects may be neglected, at least when they are not retained to be of particular importance in the specific case.
Nevertheless, it is important to note that the use of an elastic linear analysis to finished elements generally has little significance for a complex structure due to the fact that appropriate values regarding the tensional state of the masonry materials must be obtained to compare to the verification of the characteristic values of material resistance. With this type of analysis, one usually encounters tensile stresses that are not acceptable in the walls, or high compression stresses which are highly influenced by the meshing into finite elements (stress concentrations in the corners). Local verifications may therefore not be satisfactory even in conditions that in reality are safe following the local redistribution of tension in the areas under question.
In the case of palaces and villas, or buildings characterised by bearing walls and intermediate diaphragms, it is possible to fall back on equivalent framework models which allow verification at a structural level (vertical-load-bearing and lateral elements) and not of a prompt type. In this case, being that the verification conditions of a single element refers to local cracking conditions, the linear static analysis may be considered even more significant.
In the cases where seismic analysis is based on an evaluation distinguished by various local mechanisms, both for a complete evaluation of the building, as well as verification in the zones examined during the

intervention, it is possible to utilise instruments for limited analysis, in particular in the form of kinematic theorems. A linear kinematic analysis as defined in the Ordinance (Attachment 11.C), consists in the calculation of lateral load multipliers that activates a collapse mechanism and the evaluation of the corresponding seismic peak ground acceleration. For the SLU verification this acceleration is compared to the reference, reduced by means of an appropriate structural factor.

### *5.2.3 Dynamic modal analysis*

A dynamic modal analysis is performed by way of an elastic linear model (i.e. finished elements). Therefore reliability in the evaluation of behaviour in limited resistance conditions for ancient architectural masonry buildings is often limited. In fact, in the case of complex structures, a linear analysis may be usefully applied only when, in comparison with demand and capacity, a nonlinear and modest field variation emerges.

The main vibratory mode in each direction may be useful for evaluating (that which corresponds to the maximum value of the participation coefficient) and then determining a reliable distribution of forces to adopt in the linear static analysis. It is arguable, however, whether or not to consider the contribution of greater modes, which have little importance for structures characterised by nonlinear behaviour of materials because of their modest values of lateral action.

Modal analysis with a response spectrum, which presumes the linear combination of modes with rules of combination modals calibrated on framework structures, must not therefore be considered reliable, especially in the case of complex structures characterised by transformations and different construction phases.

The dynamic modal analysis may be utilised with greater confidence in the presence of easily modelled, flexible structures, for example, towers, bell towers and other structures that develop along primarily vertical lines. In these cases, the contribution of superior modes may be important. Difficulties remain however, in defining opportune structural factors and making reference to prompt verifications of the state of solicitation.

### *5.2.4 Nonlinear static analysis*

Static or nonlinear kinematic analysis consists of the evaluation of the seismic behaviour of structures (generalised links between force and shifting) and particularly the displacement capacity to the ultimate limit state. This should be compared to the displacement demand by the earthquake evaluated in terms of spectrum. Such an analysis may be performed with a model that represents the global behaviour of the construction or by utilising models of substructures (macro elements: architectonic portions which are known regarding particular collapse mechanisms), and performing local verifications.

In the case of nonlinear static analysis, the curve of structural capacity may be derived from the generalised ties between force and displacement, which are obtained through an incremental analysis. An example of this would be methods with finished elements, utilising nonlinear component connections, and if necessary, by considering the geometric nonlinearity. The analysis consists in applying gravitational loads and a lateral-force system that is scaled, maintaining the relative rapport between the same unvaried so as to monotonously augment the lateral displacement to a point of control until it reaches an ultimate limit condition. In the presence of highly irregular structures, the distribution of forces used initially may not be very significant for damaged structures. In theses cases, it is possible to fall back on an adaptive type analysis, progressively adjourning the distribution of forces.

In the case of architectural landmarks, the variety of geometry and construction systems renders the definition of prior characteristics of the distribution of equivalent static forces during a seism impossible. The analysis may, for example, be performed considering two distinct distributions of forces: a) the distribution of forces proportional to the masses; b) the distribution of forces analogous to those utilised for the static linear analysis, or proportional to the main modes of vibration in the direction of analysis (in the case of buildings, it is possible to utilise a linear height mode).

As far as the identification of ultimate displacement capacity is concerned, when the model is able to describe a structural response with resistance deterioration (*softening*). This is due to component junctions of particularly sophisticated materials and/or limited conditions of displacement of the single structural elements, and shall be defined in correspondence to a reduction of the maximum lateral reaction equal to 20%. Instead, in cases where nonlinear elastic junctions are utilised, the analysis will be brought up to a

significant displacement without the necessity of defining the ultimate displacement limit. In either situation, at the build up of movement of the control hub, the compatibility at a local level must be evaluated in terms of local crisis phenomena (beam splitting, separation between walls, ashlars, etc.).
The conversion of generalised force-displacement junctions in an equivalent bilinear system and its corresponding evaluation of the maximum response in displacement may be effectuated by means of procedures analogous to those indicated in the Ordinance (points 4.5.4, 8.1.5.4 and 8.1.6). Considering the difficulties of defining displacement up to the ultimate limit state, the ratio between the force of elastic response and the maximum force of the equivalent bilinear system cannot be superior to the admissible maximum limit, defined on the basis of the characteristics of adaptability and dynamics singular to each typology and nevertheless included between points 3 and 6.
In the cases of palaces and villas, whenever the buildings characterised by bearing walls with intermediate diaphragms, it is possible to refer to the methods proposed by the Ordinance for existing masonry buildings with the specifications indicated below in point 5.4.1 of these Guidelines.
An alternative to the method of finished elements, also in the case of nonlinear analysis, is the possibility of utilising a limit analysis, through an analysis of the kinematics of collapse, incrementally assigning the kinematic configurations varied in displacement in an increasing manner. This is called nonlinear kinematic analysis and permits the evaluation of displacement capacity of the system after the mechanism has been activated. The procedure for the determination of an equivalent bilinear system and for the evaluation of the maximum displacement response (differing from nonlinear static cases), is described in Attachment 11.C of the Ordinance.

### *5.2.5 Nonlinear dynamic analysis*

Nonlinear dynamic analysis may be utilised with nonlinear models of finished elements (or equivalent frameworks) as long as the component junctions are able to simulate not only the deterioration in stiffness and resistance at a prompt level (or of the single structural elements), but also the dissipative characteristics associated with hysteric cyclic behaviour. This analysis method cannot preclude the performance of a nonlinear static analysis with the same structural model, with the aim of determining the ultimate displacement capacity of the structure, eventually through a ductility limitation.
From an operative point of view, a nonlinear dynamic analysis presumes the use of differing groups of accelerograms (at least three) chosen for their compatibility with the response spectrum corresponding to the type of foundation terrain or otherwise determined according to points 3.3 and 3.4. It is also advisable to signal that in function of the dynamic characteristics of the structure, an adequate time frame must be selected to integrate the motion equations, eventually by utilising a convergence analysis.
It is therefore advisable to utilise this analysis method only for particular cases when the complexity of the structure and the important contribution of differing ways of vibration do not allow the assigning of a seismic response to an equivalent nonlinear system with sufficient reliability to a single grade of freedom. In these cases, a nonlinear dynamic analysis often shows an inferior displacement demand to what was estimated with a nonlinear static analysis in an evaluation.

## 5.3 Seismic assessement levels

### *5.3.1 LV1: Qualitative analysis and evaluation with simplified mechanical models*

The assessment of the seismic risk level to which a cultural heritage asset is subject to is a prerogative that cannot be disregarded for its preservation over time and for the use in safety conditions.
The evaluation of seismic safety may be led with reference to simplified methods, which are able to estimate the ground velocity on the whole when it reaches the ultimate limit state. To be even more precise, the acceleration value, compared to the peak ground acceleration value of the site, must serve only to define the seismic safety index ($I_S$), which is useful for establishing intervention priorities. Seismic strengthening interventions for mitigating risk shall be realised promptly if proven to be necessary, following an in-depth evaluation (LV2 or LV3).
The seismic safety index $I_S$ is defined here below:

$$I_S = \frac{a_{SLU}}{\gamma_I S a_g} \tag{5.1}$$

where: $a_{SLU}$ is the ground velocity which brings about the ultimate limit state; $\gamma_I$ is the coefficient of importance; S is the factor which takes into account the stratigraphic profile of the terrain beneath the foundation and any eventual morphological effects; $a_g$ is the reference peak ground acceleration of the site.
A seismic safety index with a value superior to 1 indicates that the building is able to sustain the seismic action forecast for that area; on the contrary, if the $I_S<1$, the safety of the building is inferior to what is desired, regarding the requirements requested for adequate constructions.
In paragraph 5.4, a few simplified mechanical models are proposed for evaluating acceleration to the ultimate limit state by the most common typologies and configurations.
The Ministry of Cultural Heritage and Activities has elaborated a method for the knowledge and monitoring of the state of consistency of protected architectural landmarks (Attachment A), to be realised over the next few years through a diffuse and detailed program of archiving and analysis. Being such a diffuse patrimony, the evaluation tools must be rigorous but sufficiently agile so as to be applied on a national scale. They must be based on scrupulous data gathering by way of graphic archiving forms which ascertain structural behaviour following the knowledge and formulation of a preliminary qualitative judgement regarding the level of risk (in particular, seismic risk).
The qualitative interpretation of seismic function is generally based on the registration of macro elements, in other words, by identifying architectonic parts that characterise its behaviour with a certain degree of autonomy with respect to the rest of the building. On each macro element, one or more collapse mechanisms can be identified, evaluating their greater or lesser vulnerability in relation to the presence of traditional or modern anti-seismic precautions (metal tie rods, buttresses, clamping, etc.). Greater vulnerability should also be considered when induced by transformation, deterioration and incorrect consolidation interventions. The result of this evaluation shall be expressed in linguistic format by a low, average or high vulnerability rating[10].

### *5.3.2 LV2: Evaluation of individual macro elements (local collapse mechanisms)*

This level of evaluation is applied when restoration interventions are designed which involve single parts of a construction.
The evaluation of seismic safety in the field of planned interventions on single elements can be performed by referring to local models that use specific autonomous structural components of the building (macro elements). These models can be developed by following the indications in Attachment B (nonlinear models of finished elements, limited analysis) and analysed with reference to the methods indicated in point 5.2.
In the case of local interventions, which do not modify the ascertained original function of the structure in a substantial manner, it would be particularly burdensome to impose a complete evaluation extended to the entire construction, especially when this would have to be rather articulated and the intervention would have a modest impact on the behaviour as a whole. All in all, seeing how the Ordinance requires that the acceleration of collapse of the whole building be calculated before performing any strengthening intervention, in these cases the total evaluation of seismic safety can be estimated by utilising the tools of Level 1 evaluation (LV1).

---

[10] In a purely exemplified manner, in the case of particular buildings or when their typology has not already been used for simplified modelling here below and cannot be defined by equivalents, it is possible to associate a qualitative level of vulnerability with a value of acceleration to the limit state as $a_{SLU}$; in the case of a building with coefficients with an importance rating of 1, placed on foundation terrain ranked as type A, and in the absence of topographical amplification effects, the values of the safety index can be obtained by using the seismic zones here below according to the seismic zone where the area in question is located (it should be noted however, that it is advisable to obtain the reference ground acceleration from a specialised danger map – see point 3.2):

- High vulnerability $a_{SLU}$ 0.1 - 0.2 g ag=0.35 g (zone 1) $I_S$ 0.34 - 0.57
- Average vulnerability $a_{SLU}$ 0.2 - 0.3 g ag=0.35 g (zone 1) $I_S$ 0.57 - 0.86
- Low vulnerability $a_{SLU}$ 0.3 - 0.4 g ag=0.35 g (zone 1) $I_S$ 0.86 - 1.14
- High vulnerability $a_{SLU}$ 0.1 - 0.2 g ag=0.15 g (zone 3) $I_S$ 0.67 - 1.33
- Average vulnerability $a_{SLU}$ 0.2 - 0.3 g ag=0.15 g (zone 3) $I_S$ 1.33 - 2.00
- Low vulnerability $a_{SLU}$ 0.3 - 0.4 g ag=0.15 g (zone 3) $I_S$ 2.00 - 2.67

In the definition of macro elements and collapse mechanisms, which may interest the zone subject to intervention, it is necessary to consider the eventual presence of progressive damage states (especially if of seismic origin) and the knowledge of the behaviour of similar structures (deduced from systematic surveys of post-quake damage).
A kinematic analysis, whether linear or nonlinear, generally represents an effective tool for facilitating such evaluations. The obtainable results, however, may be excessively cautious if they do not consider the diverse construction techniques that determine the real behaviour of a building: the presence of tie rods, clamping between orthogonal walls, masonry texture, and other conditions that sustain the horizontal structures.
For each macro element analysed, the comparison between acceleration at the limit state before and after the intervention allows a judgement regarding the ranking of the improvement and highlights the uselessness of certain interventions in cases where the margins of strengthening were modest with respect to the negative impact of the intervention in terms of preservation. Moreover, by considering the maximum reference ground acceleration of the site, it is possible to effectuate only necessary interventions. In fact, in elements where the acceleration to the limit state is already superior to the aforesaid, there is no need to perform seismic strengthening interventions to that part.

### *5.3.3 LV3: Global evaluation of seismic response of the building*

This level of evaluation considers the seismic safety of the entire construction, whenever the ground acceleration brings the construction to the ultimate limit state whether as a whole or significant individual elements (macro elements). The level LV3 must be adopted when interventions that modify the ascertained end use of the construction and nevertheless whenever the restoration deals with strategic types of buildings because of their social importance. Seismic safety levels must be ascertained in a reliable manner.
The complete verification of the seismic response of a building does not necessarily require the creation of a global model of the construction, but it is possible to proceed with the division of the structure into macro elements, as long as the distribution of seismic actions between the various structural systems by means of stiffness and junctures between the same are also evaluated. Such distribution can also be performed in an approximate manner as long as the equilibrium between the total of lateral actions is guaranteed. The evaluation can therefore be performed with the same tools used for level LV2, but in a systematic manner for every element of the construction.
By comparing the values obtained by the various macro elements, some strengthening interventions will be proven useless when: a) the margins of improvement are modest compared to the impact on the preservation of the building; b) excessive safety is created by certain macro elements compared to others.

## 5.4 Evaluation models by structural types

### *5.4.1 Introduction*

The concept of typology is poorly suited to historic buildings that should be considered unique historic construction elements, due to the manner they were conceived, realised and transformed over time. All in all, it is possible to acknowledge certain recurring characteristics in most historic constructions. Therefore, it may be useful to give examples of what is mentioned in the previous paragraphs in order to clarify the significance of theoretical concepts and methodologies and apply them to realistic situations.
In this chapter, certain specific examples have been furnished for the analysis and evaluation of seismic response in the most widely diffused types of protected buildings. Simplified mechanical models are proposed (LV1) for the verifications that should be performed on the entire patrimony of protected cultural heritage on a territorial scale, as an end to creating preventative risk evaluations. It should be noted how in a pervading prudence to the manner in which such models should be adopted, particular attention should be paid to the comparison of the typologies outlined in point 5.4.3 (*Churches, places of worship and other structures with large halls without intermediate diaphragms*), which being dealt with on a statistical basis, cannot exhaust the great typological diversification in which they are articulated.

### *5.4.2 Buildings, villas and other structures with bearing walls and horizontal diaphragms*

This structural typology refers to constructions that are sometimes developed in a complex planimetric manner, composed of a system of external and interior bearing walls, placed in various directions, and a system of intermediate diaphragms, which perform a connecting function. An evident analogy exists with what is generically known in building terminology and ordinary unprotected buildings. The complete modelling of buildings and villas therefore can be generally performed utilising the same global models detailed in the Ordinance for existing masonry buildings. In many cases, it is the care in construction itself, the quality of the materials and the regularity of the structural systems, aspects that usually characterise these constructions, that renders the adoption of an equivalent framework model realistic.

For the description of masonry walls with an equivalent framework, refer to the indications outlined in the Ordinance. It is evident that the validity of each specific indication must be verified with reference to the individuality of the historic building in question. For example, in the presence of a "noble" floor, characterised by especially high ceilings that vary greatly with respect to the others, the formula approximated for the calculation of the exact vibratory period cannot guarantee a reliable result. A more accurate evaluation is advised in these situations with either approximate methods or an actual specific analysis model. The same thing can be verified in the presence of loggia or porticoes, which involve a significant portion of a floor. In these situations, the same modelling of a framework equivalent may be rather imprecise. In these cases, a global analysis may be useful for a total evaluation of the safety of the construction, but should not exclude a detailed modelling of the loggia and porticoes by a local model of macro elements.

While on the subject of verification methods suggested for individual elements, it is advisable to consider that the indications for ordinary buildings are in some cases overly cautious, because they are dictated by limited experimental knowledge. For example, the resistance model for coupled horizontal masonry beams (zones set between the openings of two floors) does not consider the tensile resistance that is realised in masonry vertically by way of the mesh of the blocks. Alternative models of the behaviour of paired beams can be adopted as long as they are adequately justified.

Another determining aspect is the definition of the ultimate displacement for each element, which according to the Ordinance is a fraction of its height. The values suggested have been verified by experimentation within a certain field of possible variations, but this does not mean that it is correct to extrapolate these rules for every possible situation (for example for very low masonry piers or in coupled beams in the presence of small openings, the suggested values are certainly too low). Also in this case, it is possible to adopt alternative values when justifiable.

In the case of horizontal weight bearing diaphragms formed of vaulted elements, based on the type of vault, the characteristics of the materials, their thickness and the type of junctions imposed, an advisable value could be established for the stiffness to attribute to the equivalent diaphragm. For the latter, elastic linear behaviour may be hypothesised as long as it is defined as an ultimate angular deformation on the floor based on the type of vault.

In the case of particular kinds of structures, which are not easily attributed to an equivalent framework scheme, an appropriate model must be adopted. A possibility is to create a global structural model of the finished elements (for example, it must be able to establish the limited resistance to compression and traction, the deterioration of the mechanical characteristics in nonlinear phases, and eventually also the energetic dissipation that follows certain cyclic actions).

In particular, in the presence of certain architectural elements (large arches, loggia, cloisters, etc.), the total modelling of the building may be performed by an approximate schematic model, and performing verification on local detailed models for the substructures.

If the building is not isolated, but proves to be partially attached or forms part of an aggregate, the interactions with the other constructions should be taken into consideration, according to the position where unsatisfactory results are located (Key or corner buildings) or favourable (interior buildings), through the application of added seismic forces, that could be transmitted to adjacent constructions. Otherwise, horizontal boundaries may be inserted to calculate the appropriate stiffness (useful suggestions are found in point 11.5.4.3.2 of the Ordinance).

In the seismic behaviour of palaces and villas, the analysis of local mechanisms has proven to be of fundamental importance and should never be substituted by global analysis. Historic buildings, also when realised with high quality materials and techniques, are often not found to have systematic junctures at floor level (tie rods or stringcourses). Moreover, the criteria of geometric proportion which are adopted (distance

between interior walls, distance between openings from the corners, etc.) is not always sufficient for preventing every possible local collapse mechanism. On the basis of direct observation of the building or by considering analogous situations (found after seismic events to similar structures), potentially activating mechanisms must be identified in the construction in order to evaluate its seismic vulnerability. A possible tool is the limit analysis of equilibrium, and in particular the procedure formulated in Attachment 11.C of the Ordinance, according to the methodologies of linear or nonlinear kinematic analysis.
The analysis of local mechanisms can however be performed also on nonlinear models to finished elements, by way of an incremental analysis up to collapse.
A possible simplified mechanical model for palaces is suggested in the next paragraph.

**Simplified mechanical models (LV1)**

In the case of palaces and villas which do not show particular construction typology, a simplified mechanical model is shown below, which allows a quantitative evaluation of the collapse acceleration, in the hypothesis that this verifies wall cracking on the same floor, in the environment of a total behavioural model of the building. When buildings are shown to be more vulnerable with respect to certain local mechanisms (for lack of junctures), compared to global behaviour, the lateral acceleration which brings those macro elements of the building to the ultimate limit state must be evaluated, and then compared to what was obtained in the model illustrated here below.
The ground acceleration that leads to collapse conditions is calculated by:

$$a_{SLU} = \frac{qF_{SLU}}{e^* M\,C(T)} \qquad (5.2)$$

Where:
- $F_{SLU}$ is the shear resistance of the building;
- q is the coefficient of the structure which can be assumed to equal 3, for buildings of regular height and 2.25 in other cases having chosen an over resistance factor equal to 1.5;
- M is the total of the seismic mass;
- e* is the fraction of participating mass according to the means of collapse;
- C(T) is the normalising spectrum obtained by the ratio between the range of elastic response (point 3.2.3 of the Ordinance) and peak ground acceleration which takes site characteristics into consideration ($a_gS$).

The shear resistance of the building is obtained as the minor value between those found according to two perpendicular directions, usually chosen according to the prevalent axis of the bearing walls. The model consists in considering the bearing wall panels for each direction and hypothesising that the collapse occurs when the average tangential tension of the masonry materials reaches a certain amount of shear resistance, considering, for example, the direction "x" and a generic floor of the building:

$$F_{SLU,xi} = \frac{\mu_{xi}\xi_{xi}A_{xi}\tau_{di}}{\beta_{xi}} \qquad (5.3)$$

In which:
- $A_{xi}$ is the area resistant to shearing of the walls on the same floor placed according to direction x (it is advisable to also consider panels which are inclined $\alpha$ including those between ±45°, considering an effective area subtracted from the coefficient $\cos\alpha$);
- $\beta_{xi}$ is the coefficient of in-plane irregularity of the same floor, linked to eccentricity $e_{yi}$, of the centre of the stiffness with respect to the barycentre of the masses (whose entity can be estimated), and to the distance $d_{yi}$, between the barycentre of the rigidity and the walls in direction x plus external ones:

$$\beta_{xi} = 1 + 2\frac{e_{yi}}{d_{yi}} \leq 1.25 \qquad (5.4)$$

When all of the bearing walls have been ascertained, the coefficient of in-plane irregularity can be evaluated more accurately. Note for the generic wall k, in direction x, the resistant area in floor plan $A_{xi,k}$ and the position $y_k$ compared to a reference system, it is possible to evaluate the barycentre of the rigidity (the sum is extended to all of the walls - $\Sigma_k A_{xi,k}=A_{xi}$):

$$y_{Ci} = \frac{\sum_k y_k A_{xi,k}}{A_{xi}} \tag{5.5}$$

The coefficient of in-plane irregularity therefore is equivalent to:

$$\beta_{xi} = 1 + \frac{e_{yi} d_{yi} A_{xi}}{\sum_k (y_k - y_{Ci})^2 A_{xi,k}} \le 1.25 \tag{5.6}$$

- $\mu_{xi}$ is a coefficient that considers the homogeneity of the rigidity and resistance of masonry piers which can be evaluated as:

$$\mu_{xi} = 1 - 0.2\sqrt{\frac{N_{mxi} \sum_j A_{xi,j}^2}{A_{xi}^2} - 1} \ge 0.8 \tag{5.7}$$

  Where: $N_{mxi}$ is the number of masonry piers in direction x, in plane i; $A_{xi,j}$ is the area of a generic pier in direction x in plane i (the sum is then extended to all of the masonry piers on the same floor - $\Sigma_j A_{xi,j} = A_{xi}$).
- $\xi_{xi}$ is a coefficient linked to the type of breakage expected in the prevalence of masonry piers on the same floor; this equals 1 in the case of shear collapse, while it may be equal to the value of 0.8 when there is collapse caused by combined compressive and bending stress (thin piers or little vertical load);
- $\tau_{di}$ is the value of the calculation of the shear resistance of the walls of the masonry piers of floor i:

$$\tau_{di} = \tau_{0d}\sqrt{1 + \frac{\sigma_{0i}}{1.5\tau_{0d}}} \tag{5.8}$$

  Where: $\tau_{0d}$ is the value of the calculation of the shear resistance of the masonry (evaluated by considering the confidence factor $F_C$); $\sigma_{0i}$ is the average vertical tension on the resistant surfaces of walls on the same floor.

The mass M to consider for the evaluation of seismic actions of collapse is associated to gravitational loads

$$M = \frac{(G_k + \sum_i \psi_{2i} Q_{ki})}{g} \tag{5.9}$$

Where: $G_k$ are the permanent loads (at their characteristic values), calculated on the entire building; $Q_{ki}$ are the variable accidental loads (at their characteristic values); g is gravitational acceleration; $\psi_{Ei}$ is a combined coefficient which takes into consideration the probability of all the variable loads present during an earthquake.

In conclusion, in order to evaluate the fraction of mass which participates in the dynamic motion e* it is necessary to first hypothesise a means of collapse. The symbol **Φ** indicates the vector that represents the displacement of the various floors according the forms assumed as the means of collapse (this cannot be the same figure used as the unitary value at the building summit). The fraction of participating mass according to the means of collapse is calculated by:

$$e^* = \frac{\left(\sum_i m_i \phi_i\right)^2}{M \sum_i m_i \phi_i^2} \tag{5.10}$$

In which (being that the sums are extended to all the floors of the building): $m_i$ is the mass on the same floor; $\phi_i$ is the lateral displacement of the same floor.

When it is possible to assume that both the height of the space between floors and the mass of the floors is substantially constant, the formulation may be simplified as follows:

$$e^* = \frac{\left(\sum_i \phi_i\right)^2}{N \sum_i \phi_i^2} \tag{5.11}$$

Where N is the number of floors.

When the means of collapse cannot be precisely defined**Φ**, it is possible to refer to two recurring collapse mechanisms in the case of buildings:

- Collapse of the same floor k – occurs when a floor is shown to be weaker than the others, so the construction cannot manage to utilise the dissipative capacity and the displacement of the other floors which remain in the elastic phase (this mechanism also occurs in the presence of stringcourses or other stiffening elements in the masonry walls at the diaphragm level); in this case, the fraction of the participating mass is shown as:

$$e^* = \frac{N+1-k}{N} \tag{5.12}$$

- Uniform collapse – occurs when the collapse shows breakage of the horizontal masonry diaphragms at floor level, and the masonry piers collapse at the base of the first level because of combined compressive and bending stress; the fraction of participating mass assumes this equation:

$$e^* = 0.75 + 0.25N^{-0.75} \tag{5.13}$$

The evaluation of collapse acceleration is performed by following the steps below:

- Ground floor collapse:

a) Calculation of the resistance according to the two in-plane directions and the identification of the direction of greatest weakness;

b) The choice between the hypothesis of collapse for weak floors (5.12) or of a uniform kind (5.13), depending on stiffness and resistance of the masonry panels between openings at the floor level;

c) Evaluation of the collapse acceleration on the ground floor;

- For the upper floors of the construction:

a) Calculation of the resistance according to the in-plane directions and the identification of the weakest direction;

b) Evaluation of the collapse acceleration assuming the hypothesis of collapse due to weak floors;

- Evaluation of the building acceleration $a_{SLU}$, as the minimum between the values corresponding to the collapse for various floors.

### *5.4.3 Religious buildings and other structures with large halls, without intermediate diaphragms*

The systematic analysis of damage done to churches during principal Italian seismic events, starting from Friuli (1976) up to the most recent ones (Lunigiana and Garfagnana, 1995; Reggio Emilia, 1996; Umbria and Marche, 1997; Piedmont, 2000; Molise, 2002; Piedmont, 2003; Salò, 2004), has shown how seismic behaviour in these kinds of constructions can be interpreted by way of their division into architectural components (called macro elements), characterised by a substantial autonomous structural response compared to the church as a whole (façades, halls, apses, bell towers, domes, triumphant arches, etc.).
Only in the cases of churches with a central floor plan, usually complete with one or more symmetrical in-plane axis and with homogeneous construction and good connections between the elements, it is important to proceed with a complete model of the construction (linear or nonlinear), evaluating, for example, the capacity curve through an incremental collapse analysis. In any case, all effects due to thrusting forces of arches vaults and coverings must be verified.
Most of the time, local verifications should be made which generally refer to various macro elements, which in turn become references for the structural verifications. On the individual macro elements, it is possible to conduct a linear or nonlinear static analysis, for example, with a model of finished elements. All in all, the kinematic methods introduced in point 5.2 and described in Attachment B (whether linear or nonlinear) have proven to be the most effective. These methods have been approved of for the evaluation of local mechanisms in existing masonry buildings (Attachment 11.C of the Ordinance). The uncertainties of the choice of mechanisms prior to collapse is a crucial point of the kinematic approach in the field of limited structural analysis, which in this case are quite limited thanks to the in-depth knowledge regarding damage mechanisms of churches derived from systematic damage surveys.
The evaluation of seismic safety as a means of choosing strengthening interventions must utilise careful interpretation of ascertained damage sustained by the church over time, both of constructional significance as well as significant construction details with respect to response to lateral displacement.
In the case of strengthening interventions following an earthquake, the identification of macro elements and their corresponding collapse mechanisms must obviously be correlated to the ascertained function through a survey of the seismic damage.
Regarding eventual preventative work, the systematic analysis of construction details (quality of the masonry, clamping, the presence of anti-seismic measures, etc.) may allow the identification of macro

elements and damage mechanisms that are easily activated. In these cases, the survey of historical events can only be an aid, due to the fact the construction may have been modified since the time the earthquake occurred.
In light of this, the necessity of performing a complete LV3 analysis (the total evaluation of the seismic response of the building) or local LV2 (evaluation of single macro elements of local collapse mechanisms), does not determine a substantial difference in approaching the difficulties of modelling.
When interventions have to be made which may influence the global response of the structure, an evaluation of ground acceleration must be performed before and after the intervention on all of the church's macro elements with regards to differing limit states. In the actual state, a preliminary evaluation of lateral seismic displacement between the macro elements must be effectuated; in the design stage, the structural modifications introduced by the strengthening intervention (masses, stiffness, junctures) may have a negative effect on certain macro elements (Correct interventional strategy which does not conserve the ascertained function consists in the proper use in a limited and controlled manner of the best resources of certain elements).
When strengthening interventions involve a limited area (for example, in accordance with restoration interventions on decorative walls), the evaluation may be limited to level LV2, because a total evaluation of the church may result to be superfluous and problematic (this occurs in particular for large and complex churches in accordance with local interventions done on single macro elements, with an aim to control the effectiveness of the intervention (safety check before and after) and its congruity with respect to onsite risk. In these cases, since an evaluation of the capacity of the entire structure is nevertheless required, it is possible to adopt a simplified model (LV1), an example of which is shown below.

**Simplified models for estimating the ground acceleration that corresponds to limit states (LV1)**

As said above, in most churches, assuming a unitary and complete structural behaviour has proven to be of little significance. Therefore, due to the overwhelming construction and typological variety of churches, it is better not to define a simplified mechanical model for evaluating seismic safety, based on a limited number of parameters as was done for building types.
However, the possibility of defining specific mechanical models remains, which are valid for each case in question or groups of buildings. An alternative to a LV1 evaluation is to fall back on the parameters used for damage and vulnerability survey forms[11], which utilise previously consolidated experiences before the drawing up of these guidelines. Peak ground acceleration corresponding to various limit states may be correlated to a numerical indicator, the vulnerability index $i_V$, obtained by way of the correct combination of assigned points to differing elements of vulnerability and anti-seismic measures.
Damage and vulnerability survey forms for churches have been utilised in seismic emergencies since 1995. The notable collection of data gathered (over 4000 churches) has allowed the establishment of a relation between seismic events and damage within the parameters of the vulnerability of churches through statistical elaborations. It is evident that an estimate assumes a purely statistical value, but this approach may be considered correct when utilised on a territorial scale as a means of establishing a list of priorities and for the better planning of more in-depth evaluations and direct preventative interventions. Moreover, the use of unitary models for evaluations of this nature permits a more objective comparison in terms of seismic risk.
The methodology considers 28 damage mechanisms associated to various macro elements that may be present in a church. With reference to vulnerability evaluations, it is necessary to survey the typological and constructive particularities that play a fundamental role in the seismic response of the building. In particular, vulnerability indicators and anti-seismic measures have been considered. In Attachment C, the 28 mechanisms have been listed along with a list of measures and vulnerability indicators which may be added to in the future with respect to singular aspects of individual churches or other buildings in the geographical area in question.
The seismic behaviour of the entire building is represented, on a statistical basis, by a vulnerability index that varies between 0 and 1, which is defined as the average behavioural weight of the different parts of the church:

[11]2nd level Church form for damage and vulnerability assessment: in the Molise Region, National Advisory Board for Research. *Protected Monuments and Earthquakes: from emergency to reconstruction*, DEI Typography from the Civil Engineers in Rome, 2005 (Chapter 4, Attachments C1, C2 andC3).

$$i_V = \frac{1}{6}\frac{\sum_{k=1}^{28}\rho_k\left(v_{ki} - v_{kp}\right)}{\sum_{k=1}^{28}\rho_k} + \frac{1}{2} \tag{5.14}$$

Where for every k mechanism: $v_{ki}$ and $v_{kp}$ are respectively the points obtained by the vulnerability survey and from anti-seismic measures (Table 5.1); $\rho_k$ is the weight attributed to the mechanism (it is set at 0 for mechanisms which were not able to activate in the church due to the lack of macro elements, while values between 0.5 and 1 are given in other cases).
During the numerous surveys performed on churches damaged by earthquakes, the level of damage sustained by each macro element was also evaluated in relation to the various damage mechanisms possible. This leads to a damage index, variable between 0 and 1 as a normalised average of local damage:

$$i_d = \frac{1}{5}\frac{\sum_{k=1}^{28}\rho_k d_k}{\sum_{k=1}^{28}\rho_k} \tag{5.15}$$

where $d_k$ is the level of damage sustained with respect to the same k mechanism (from 0 to 5).

Table 5.1. Point evaluation of vulnerability for each damage mechanism.

| Number of vulnerability indicators or anti-seismic measures | effectiveness judgement | $v_k$ |
|---|---|---|
| at least 1 | 3 | 3 |
| at least 2 | 2 | |
| 1 | 2 | 2 |
| at least 2 | 1 | |
| 1 | 1 | 1 |
| none | 0 | 0 |

From the statistical analysis of damage sustained, the probable distribution associated with various seismic intensities (probability damage matrix), varies the vulnerability index. Through a proper correlation between intensity and peak ground acceleration, it is possible to define a direct correlation between seismic input and revealed vulnerability. This allows the calculation of a ground acceleration value for each church, which corresponds to the limit state of damage (SLD) and the ultimate limit state (SLU). The functional correlations are shown below:

$$a_{SLD} = 0.025 \cdot 1.8^{2.75-3.44 i_v} \tag{5.16}$$

$$a_{SLU} = 0.025 \cdot 1.8^{5.1-3.44 i_v} \tag{5.17}$$

As an example, in table 5.2, vulnerability indices are shown which correspond to ground acceleration to SLD and SLU equal to what is set for the four seismic zones of Type A terrain. Obviously, in each situation, the index of seismic safety is more than one when the vulnerability index is inferior to the values in the table. From this same table, it emerged that a church in zone 4 on average has proven to be protected with respect to SLU, while in zone 1, there are only churches with low vulnerability ($i_V$<0.18). It should also be noted that in the case of low intensity quakes, these structures are more sensitive with regards to SLD than to SLU.

Table 5.2. Maximum values of vulnerability indices for verifying diverse seismic zones.

| | Zone 1 | Zone 2 | Zone 3 | Zone 4 |
|---|---|---|---|---|
| SLU | 0.18 | 0.34 | 0.6 | 1 |
| SLD | 0 | 0.11 | 0.37 | 0.91 |

Along with the evaluation of the total safety index of the construction obtained as shown above, it is advisable to note the presence of elements or mechanisms with particularly high vulnerability.

### *5.4.4 Towers, bell towers, and other tall and slender structures*

This type of construction is generally distinguished on the basis of its prevalently vertical development and constitutes an important collection of Italian constructive patrimony.
Seismic behaviour of this type depends on certain specific factors: the width of the structure, the grade of clamping in the walls, the eventual presence of adjacent structures in the lower portions which may create horizontal limits, the presence of thin architectural elements at the summit of the structure (steeples, bell towers, crenulations, battlements, etc.) or nevertheless vulnerable (belfries). Moreover, vulnerability is influenced by the presence of damage states of other natures, caused for example by vibrations induced by the bells or foundation problems.
Thinness is a highly variable parameter. Indeed, rather stocky towers exist (i.e. look-out towers or medieval bastions of defence) and towers that are very thin. If the first can be considered as massive constructions, due to their great wall thickness formed by dry wall masonry, the latter can be considered to be mono-dimensional structures with shelf-like behaviour.
Wall clamping in masonry structures which develop along primarily vertical lines works to guarantee that the clamping acts like a walled-in shelf at the base, with stiffness associated to the entire masonry section (preservation of flat sections) and not like a group of distinct walls. The traditional techniques which guarantee good clamping are: interlocking corners; the presence of reinforcement rings and tie-rods; the presence of well-connected diaphragms. Moreover, the effect due to thrust must also be carefully evaluated in the case of vaulted masonry arches.
Quite frequently, towers and bell towers are in contact with other lower structures. Some usual cases are: towers built as part of or next to churches; towers incorporated in various ways within the urban setting; towers built into city walls. The presence of horizontal diaphragms at differing levels can deeply mutate the behaviour of the structure. On one hand, by limiting the effective thinness, and on the other by creating localised stiffness and points where a concentration of forces is possible (observation of damage has generally demonstrated that these situations are the cause of even significant damage). It should be remembered that such junctures are often different depending on the two main directions of the horizontal plane. In these cases, the verification should be performed starting form the point of separation, being careful to consider the effect of these junctures on the form of the collapse mechanism and the amplification of seismic action at that height of the structure. These situations are often characterised by noteworthy vulnerability.
In bell towers, the belfry can be a particularly vulnerable element, due to the ample holes that are often composed of thin and light load bearing pillars with shear fissures due to shifting. Analogous considerations must be made for thin and lopped elements found at the tops of towers. Their vulnerability is due first to their modest vertical load bearing ability (associated only to weight) that guarantees limited stabilising effect with respect to overturning. The amplification of seismic motion has an even more critical effect when verified in the higher parts of the construction. The observation of damage has in fact demonstrated how similar towers behave in very differing ways even when the seismic action at the base of the tower was the same, and therefore the diverse interaction between earthquakes, foundation terrain, structure and superstructure.
Considering the minor geometric and constructive complexity of these kinds of structures compared to those analysed in the preceding paragraphs, all of these aspects can usually be studied with adequate precision by way of reliable, detailed, structural models. In this case, we retain that with due caution even linear models may supply useful and reliable indications because the redistribution of forces in a primarily isostatic structure is always modest. This allows the use of dynamic analysis, in particular modal types are of primary importance for understanding the aspects of amplification of the motion described above.
The relative clarity of the structural scheme and the behaviour of these types of structures allows the creation of simple models in many real situations and damage and collapse mechanisms in some typical cases.

**Simplified mechanical models (LV1)**

Damage and collapse mechanisms for these kinds of structures are various and depend on geometric variables like slenderness as well as constructive characteristics (masonry quality and block interlocking). In the case of rather squat towers, shear fractures are found, while in the presence of shoddy masonry quality, vertical fissures have been verified which start from the belfry and tend to divide the structure into parts.

The development of simplified mechanical models capable of analysing these mechanisms for a general overview of the actual situation is not possible. Therefore, it is necessary to rely on specific verifications albeit approximate. For a quantitative evaluation with simplified mechanical models, however, it is possible to refer to collapse for flexion pressure, or by considering towers as shelves, solicited by lateral forces in addition to their weight which may be subject to crises in a generic section for crushing in the compressed zone, after choking due to non-resistance under traction.

The verification of combined compression and bending stress of thin masonry structures is done by comparing the calculated agent momentum with the ultimate momentum of calculated resistance assuming the masonry is not resistant to traction and conducting an opportune nonlinear distribution of compression.

The verification should be performed according to the two main directions of inertia of the section at differing heights because it is not possible to identify the critical sections in advance due to the presence of tapering in the thickness of the masonry and weaknesses because of wall openings. To these ends, the structure should be divided into *n* sectors of uniform geometric characteristics, and the verification should be performed in correspondence to each section change. The verification of each section should be conducted according to the two main directions because in the direction of greatest stiffness the period of principal vibration being inferior may generate a higher seismic response.

When structures have squared rectangular sections, in the hypothesis that normal forces are not superior to $0.85f_d$as, the ultimate resistant momentum at the base of the same sector may be calculated as:

$$M_{u,i} = \frac{\sigma_{0i} A_i}{2}\left(b_i - \frac{\sigma_{0i} A_i}{0.85 a_i f_d}\right) \tag{5.18}$$

where:

- $M_{u,i}$ is the momentum which corresponds to collapse due to combined forces of the section being analysed;
- $a_i$ is the perpendicular side of the direction of the seismic action considered in the same section being analysed, depurated of any eventual openings;
- $b_i$ is the side parallel to the direction of the seismic action considered in the analysed section;
- $A_i$ is the total area of the section under analysis (when quarried sections of constant thickness are equal to $2s_i(a_i+b_i-2s_i)-B_i$, with $s_i$ as the masonry thickness in the section and $B_i$ is the area of the holes present);
- $\sigma_{0i}$ is the average normal tension of the section being analysed ($W/A_i$, with W being the weight of the structure which is present above the section under analysis);
- $f_d$ is the resistance to calculated compression of the masonry (taking into account the confidence factor $F_C$).

The calculated momentum agent may be evaluated by considering a system of forces distributed along the height of the structure, assuming a linear form of displacement. The force to apply in correspondence with the barycentre of each ashlar is given in the following formula:

$$F_i = \frac{W_i z_i}{\sum_{k=1}^{n} W_k z_k} F_h \tag{5.19}$$

where:

- $F_h = 0.85 S_d(T_1) W/g$;
- $W_i$ and $W_k$ are the weight of sectors i and k respectively;
- $z_i$ and $z_k$ are the heights of the barycentre of the sectors i and k with respect to the foundation;
- $S_d(T_1)$ is the response spectrum, function of the first period $T_1$ of the structure according to a given direction;
- $W=\Sigma W_i$ is the total weight of the structure;

- g is gravitational acceleration.

The result of the seismic forces acting in the same section is obtained by:

$$F_{hi} = \frac{\sum_{k=i}^{n} z_k W_k}{\sum_{k=1}^{n} z_k W_k} F_h \tag{5.20}$$

The height $z_{Fi}$ to which force $F_{hi}$ is applied is evaluated by the relationship between:

$$z_{Fi} = \frac{\sum_{k=i}^{n} z_k^2 W_k}{\sum_{k=i}^{n} z_k W_k} - z_{i*} \tag{5.21}$$

where:
- $z_k$ is the quotient of the barycentre of the mass of the same sector k with respect to the base which has a weight of $W_k$;
- $z_{i*}$ is the quotient of the same i section being verified compared to its base;

Setting as equal the ultimate momentum and the design momentum:

$$M_{u,i} = F_{hi} z_{Fi} \tag{5.22}$$

it is possible to obtain the critical value of the response spectrum:

$$S_{d,SLU,i} = \frac{M_{u,i} g}{0.85 W z_{Fi}} \frac{F_h}{F_{hi}} \tag{5.23}$$

The value of peak ground acceleration that corresponds to the ultimate limit state in the same i section is valued at:

$$a_{SLU,i} = \begin{cases} 0.4\, q\, S_{d,SLU,i} & T_1 \le T_C \\ 0.4\, q\, S_{d,SLU,i} \dfrac{T_1}{T_C} & T_1 > T_C \end{cases} \tag{5.24}$$

where:
- q is the structural factor that when subtracted from a more accurate evaluation, may be assumed analogous to that of buildings, thus equal to 3 when the regular structure is in elevation, or reduced to 2.25 in the presence of brusque changes in stiffness along the height or when there are attached adjacent structures;
- $T_C$ is the typical response spectrum period, defined in point 3.2.3 of the Ordinance (assuming that these types of structures never have a period inferior to $T_B$ or superior to $T_D$);
- $T_1$ is the first period of vibration of the structure, which can be evaluated by means of:
  a) a modal analysis of a shelf-type equivalent model;

b) iterative algorithms that consider the variation of the mass and the stiffness along the height (utilising the values of cracked elastic forms for the materials), starting from an opportune assumed modal form;

c) simplified formulas when retained sufficiently reliable. In the case of structures which develop along mainly vertical lines, the first period of vibration in the elastic phase can be obtained by dynamic measurements with environmental noise (or forced, for example when induced by bells);

For the evaluation of the behaviour at the ultimate limit state, the period in the cracking phase must be utilised, which can be obtained by multiplying the value of the identified period of the dynamic measurement by a coefficient of between 1.4 and 1.75.

Acceleration to the ultimate limit state $a_{SLU}$ for structures is given by the minimum $a_{SLU,i}$ found.

In the case of towers or bell towers which attach to other lower structures, it is also necessary to evaluate the section placed at the height where they separate, assuming an opportune modal form differing from linear ones, both due to the estimate of the vibratory period as well as for the calculation of the height $z_{Fi}$ to which force $F_{hi}$ is applied.

### *5.4.5 Masonry bridges, triumphal arches and other structures with arches*

Arched (or vaulted) elements are quite diffused in the monumental heritage both as individual structures (triumphal arches) as well as the characterising feature of more complex structures (masonry bridges).
Intuition of the seismic behaviour of arches with respect to lateral actions is not to be taken lightly, due to the lack of an observational system of post-earthquake damage and analysis methods which are not diffused within the professional field. Certain aspects regarding the response of single arches can therefore be useful for understanding more complex structures. As far as vertical loads are concerned, the arch-pier systems (or barrel vaults attached to walls) go into crisis for loss of equilibrium; if the loads are primarily on the keystone, five hinges are formed, one at the keystone, with an opening for an intrados crack; the five hinges subdivide the structure into four blocks, which can be assumed to be rigid. In the case of lateral seismic action, the system cracks in an asymmetrical manner, with the formation of four hinges, two in the arch and two in the piers, with an intrados crack slightly shifted compared to the keystone and an extrados one towards the rear. In both load conditions, the sliding mechanisms are rare, due to the fact that friction between the ashlars creates an effective contrast, normal forces being always orthogonal to the plane of contact between the ashlars themselves.
In the case of collapse studies for individual arches, it is important to evaluate (in addition to the presence of static loads) the real structural geometry: in fact, above the settings, a structural masonry element is often present composed of masonry or conglomerate. This element is not a mere filler, but if adequately constructed, is an element which reduces the vulnerability of the structure. In fact, cracks generally form above this element (which, in turn reduces the span of the arch). Models that do not consider these constructive situations are found to be too cautious concerning collapse.
Moreover, arched structures prove to be very sensitive to damage in cases where differential movements occur at the settings. Seismic action, in the cases of greatly lighted arches (bridges) may induce motion which is not synchronised at the base of the piers, caused both by wave propagation in the terrain as well as by the effect of differing local amplification of motion when there are different terrain conditions at the base of the piers; in all of these cases the earthquake imposes, instant by instant, different lateral displacement on the arch and consequently, cracking may occur.
Masonry arch bridges are elements of great importance from a historic and cultural standpoint. The architecture of their infrastructures is particularly important also from a strategic point of view, often being still utilised for vehicle and rail traffic. The seismic behaviour of theses structures differs according to typology.
In the cases of bridges across the river in flat regions, the piers are generally reduced in height and have low rise arches. The longitudinal response of the structure primarily linked to the modal form where the piers behave like cantilever (in the 1$^{st}$ mode of vibration) do not present particular problems due to their squat shape and limited displacement at the their top. As far as transversal actions are concerned, the piers, which are not very slender, may be subjected to shear cracks and, in the case of excitation on higher modes (alternating vibration of the piers) diagonal cracks may occur in the vaults.
In the case of bridges with only one arch of long span, as long as there are no underlying problems with the abutments, the response is usually influenced by the vertical component of the seismic force.
The viaduct type structure, given the notable slenderness of the piers, has proven to be sensitive to both longitudinal as well as transverse forces. In the former, if the piers differ in height they may vibrate with different periods and thus create shifting at the settings of the arches with consequent cracking. In the latter, the displacement at the top of the piers may be counter-phased so the upper part of the structure distorts; thus the response depends on the stiffness of the same compared to the vertical support elements.
In the case of viaducts with multiple arches, the piers may be set into terrain with varying geo-mechanical characteristics. The different properties of the terrain, the depth of the layers and topographical effects produce amplification or deamplification phenomena of seismic motion. The action at the base of the different piers therefore takes on a spatial variable, which translates into differentiated excitation (unsynchronised motion).
Finally, local collapse mechanisms are possible in the tympana, above the arches that behave exactly like filled containment walls (ballast), and exercise a static pushing force that augments in the presence of an earthquake. Analogously to masonry buildings, these mechanisms are often those that occur first, also for

low seismic forces. The observation of damage produced on the occasion of recent earthquakes in Italy has substantially demonstrated the activation of only these mechanisms. The verification of local mechanisms of single portions of masonry for out-of-plane actions is therefore essential also for the evaluation of the seismic safety of masonry arch bridges. Procedures analogous to those proposed in the Ordinance for buildings could be performed, considering incremental action due to the position of the tympanum within the construction.
Finite element modelling may provide a detailed evaluation of a masonry arch bridge; however, it presents many difficulties in the evaluation of the elastic moduli to be attribute to certain parts of the structure. For example, the in-fill represents a significant amount of the structure in terms of volume and therefore the response shall be greatly influenced by the parameters attributed to such element.
For a more reliable evaluation of seismic capacity in the ultimate limit state, it is necessary to adopt nonlinear constitutive laws. Since collapse mechanisms related to arches and piers are mainly governed by the formation of hinges, with cracks in correspondence of mortar joints, the nonlinear elastic constitutive law of NTR (No Tensile Resistance) solids seems to be adequate even if one should not forget that being isotropic, it does not consider the orientation of the mortar joints between the elements. Such model however does not allow correct description of the tympanum response, characterised by damage due sliding with friction and by the in-fill.
The collapse of an arch bridge subjected to seismic actions may be evaluated by way of the limit analysis of equilibrium. The static approach has proven to be rather complex from an operational point of view. In fact for each increment of the lateral seismic force, it is necessary to find a new equilibrate thrust line in each point of the structure. The kinematic approach, although simpler to apply, presents many difficulties, mostly in multi-span bridges because the identification of kinematic collapse mechanisms depends on many hinges, and the response may prove to be sensitive to their positioning, especially in the archways.

# 6 SEISMIC IMPROVEMENT CRITERIA AND STRENGTHENING INTERVENTIONS

## 6.1 Strategies for the choice of improvement interventions

Strengthening interventions to structures, at times to reduce seismic vulnerability, are to be evaluated within the general context of building preservation. The choice of the strategy and technique of intervention, as well as its urgency, depends on the results of the preceding evaluation phase.
The main objective is always the conservation not only of the materials but the ascertained behaviour of the structure as well, whenever this does not compromise the safety of the historic building. In this sense, aspects linked to interventions for installing technological systems should also be carefully evaluated, privileging the solutions that limit or exclude the insertion of systems within structural elements.
Interventions should generally deal with a single part of the building, containing as much as possible the number and nevertheless avoiding significant alterations to the original distribution of stiffness in the structural elements. The execution of strengthening interventions to limited portions of the building should nevertheless by evaluated and justified within the framework of an indispensable vision of the whole, taking into consideration the effects of variations in stiffness and strength the elements may be subjected to.
Design of interventions must guarantee the preservation of the architecture in all of its declinations, especially evaluating any eventual interference to decorative apparatus.
An intervention must be realised only after ascertaining the benefits that can be obtained and the impact to the historic construction. In particular, all operations of demolition and substitution or demolition and reconstruction should be avoided, utilising only interventions that integrate the existing structure without radically transforming it. In emergency situations, these conditions may be partially disregarded, adopting however provisional solutions that produce minimum permanent changes.
Safety evaluation and clear understanding of the structure, must be the bases of the decisions and choices made for intervening. In particular, interventions must have a good rapport 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, when speaking of the prevention of seismic damage, simple maintenance often helps avoid later major transformative interventions.
The choice 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 interventions that transform the building in a non-permanent way; new materials, which are the result of technological innovations, should be evaluated in the light of compatibility criteria and their durability over time in relation to historic materials.
As much as possible, interventions should respect the original concepts and techniques of the structure 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 to 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 insure their real effectiveness and avoid damage that could worsen the mechanical characteristics of the masonry or the functionality of the structural elements. As much as possible, the proposed interventions should be controlled throughout the work's progress. All necessary documentation describing the realized works should become an integral part of the final report describing the intervention. All control and monitoring activities must be documented and kept as part of the history of the construction.
The strategy of the intervention may belong to one of the following general categories or a combination of them:

- Reinforcement of part or all of the resistant elements as an ends of selectively increasing resistance, stiffness, ductility or a combination of these (always paying careful attention to induced modifications to the structural scheme);
- Insertion of new elements which are compatible with existing ones as an ends of eliminating local vulnerability of certain parts of the construction and improving the overall functionality in terms of resistance or ductility;
- Introduction of passive protection by means of dissipative braced structures and/or base isolation (accurately considering all possible fallout on the preservation and in particular the presence of archaeological substratum);
- Reduction of the mass (with all due precaution);
- Limiting or changing the use of the building (in this case, a compatibility verification is obligatory for the planned urban transformation in the projects in realisation and changes to the building's end use).

In general, interventions differ due to:

- *Expansions*: Interventions limited to a few elements; interventions extended to the entire structure
- *Ascertained and obtained static (and dynamic) behaviour*: Interventions that do not modify the actual static behaviour; Interventions that, despite modifying the actual static behaviour, are coherent with the functionality of the structural typology; Interventions that modify the original static behaviour.
- *Invasiveness and reversibility:* An intervention is considered invasive when it permanently modifies the resistant elements; it is the contrary of a reversible intervention, which integrates the resistant elements and/or stress state without transforming the original structure in any permanent way.
- *State of supplied co-action*: Passive interventions which do not supply a state of co-action to the original elements; Active interventions which introduce states of stresses through self-balancing actions.
- *Architectural Integrity*: Interventions which are able to preserve not only the material quality of the construction but also its aspect
- *Durability and material compatibility*: Interventions that last also in relation to the chemical-physical characteristics of the historic materials.

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

## 6.2 Influence of equipment upgrade interventions

Often strengthening interventions to existing buildings stem from needs that may not be specifically structural. Examples of this are updating wiring and plumbing equipments linked more or less to adapting to new laws and regulations. Nearly always, these interventions interact with structural parts of the building, and frequently, if not adequately calibrated, bring about a sizable reduction to the resistance of the elements involved and sometimes transform the construction's functionality.
Therefore, it is essential that each time there is interaction with elements of structural importance (both compared to vertical loads as well as lateral ones) the impact of the intervention on the resistance capacity is evaluated as well as the eventual repercussions on the elements that interact with them or on the structure

as a whole. The evaluation of a seismic safety index must thereby refer to the eventual modified structure after interventions even when not obviously structural.
It is therefore opportune to avoid interventions that bring about a reduction to the resistance capacity of the elements or reduce the effectiveness of their junctures. Whenever this is not possible, it is necessary to evaluate the impact such interventions have on the global behaviour of the construction, and adopt adequate countermeasures by means of specific interventions that act to balance any negative effects. Nevertheless, they must also be compatible with the needs of preserving historic landmarks.
Below, a few examples of possible interventions are shown which may reduce the resistance capacity of a structure:

- Cuts in the horizontal structures, either on the entire thickness (formation of atriums) or in tracks (passing equipment tubes through the flooring);
- Horizontal cuts in the masonry, where it is not possible to limit the extension and depth and the reduced resistance capacity of the elements involved must be taken into consideration;
- Vertical cuts in the masonry which, due to their position, in some cases are may reduce not only the resistance of the elements, but also the junctions between masonry walls;
- Any work that worsens the structural configuration in regards to seismic actions, either in terms of constructive details or on the global configuration (i.e. demolition near wall corners).

All of the above is to be intended both to vulnerability with regards to local mechanisms, as well as the behaviour of the construction as a whole. In the latter, the impact of all the interventions connected to the updating of equipments must be evaluated wherever they are performed in different parts of the building even when not contemporaneous.

## 6.3 Techniques of strengthening interventions

### *6.3.1 Introduction*

Below, some general indications for the choice of seismic strengthening interventions for masonry buildings are shown, with reference to techniques commonly used today. There is usually more than one possible intervention for each kind of pathology or form of vulnerability, with distinct characteristics in terms of effectiveness, invasiveness, reversibility, compatibility, durability and cost.
The choice of the most suitable solution is the primary concern of the project and must be made after careful examination of the specific situation and verification of effectiveness. To this aim, it is advisable to provide preliminary feasibility trials for the intervention and program post-intervention surveys in order to certify the final outcome.
The possibility of falling back on provisional methods should not be overlooked, which due to their striking 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 the necessity of performing a seismic safety evaluation has emerged. Obviously, techniques that have not been cited herein should not be excluded including innovative methods or particular solutions that professionals have identified as adequate for their specific cases.
In any case, no intervention should ever be considered primarily non-structural or completely lacking in any effects on structural behaviour.

### *6.3.2 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 <u>insertion of tie-rods</u>, 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.

### *6.3.3 Interventions to reduce thrust of vaulted arches 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, which 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.

### *6.3.4 Interventions to reduce excessive flexibility of horizontal diaphragms and their consolidation*

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.

### *6.3.5 Interventions for roof coverings*

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.

### *6.3.6 Interventions for increasing the strength of masonry elements*

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.

Recent experimental tests have shown 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.

### *6.3.7 Pillars and columns*

Since pillars and columns are essentially designed to support vertical loads with modest eccentricity, strengthening interventions should be designed in order to:

- Reconstruct the initial resistance to normal forces by way of provisions such as ringing and plugging; in some cases the use of bonding with resin may be acceptable;
- Eliminate or nevertheless contain lateral thrust by way of strengthening such as the insertion of tie-rods in the presence of arches, vaulted arches and coverings, or where necessary, the realisation or reinforcement of buttresses;
- Reconstruct or realise appropriately stiffened junctions as a means of transferring lateral actions to the masonry elements with the greatest resistance.

The insertion of metallic rods in axis to the column in order to take over the vertical load, or pre-compressed vertical tie-rods in order to give greater resistance to shear flexion should be avoided unless no other viable solutions are available as demonstrated by technical specifications.

### *6.3.8 Interventions on non-structural elements*

For the evaluation of seismic vulnerability of non-structural elements (cornices, parapets, or chimneys), possible amplification of the acceleration at different heights of the building should considered as well as the dynamic interaction between those elements and the structure. Generally, the experience of builders, consolidated over the centuries, and inspected by the passing of time, must be kept in mind for judging the safety of these elements, especially those which do not show particular problems in their connection to the structure (cracking, rotation, etc.).
Where there are problems, it is best to intervene to improve the shifting capacity prior to the ultimate limit state by way of lateral restraints or by widening the support base, and eventually improving its connection to the structure, keeping in mind that a variation of the dynamic properties may augment seismic action on the element.

### *6.3.9 Interventions on foundations*

The inadequacy of the foundation is rarely the sole or primary cause of damage observed following an earthquake. It is possible to omit strengthening interventions to the foundation structures as well as the relative verifications whenever the following conditions are found:

- No significant displacement is present which can be attributed to foundation sinking, and it has been ascertained that no displacement of this nature had ever been verified in the past;
- The designed interventions on the structure in elevation do not bring about any substantial alterations in the static scheme of the building;
- The same interventions do not cause relevant changes to solicitation transmitted to the foundations;
- Overturning phenomena due to seismic activity has been excluded in the construction.

When research and analysis bring to light the necessity of strengthening interventions for the foundations, it must be aimed at creating the maximum uniformity for the support conditions as a means of obtaining the most uniform distribution possible under contact pressure. To such an ends, generally interventions which widen the base of the foundation is chosen over those which use small dimensioned stakes or other localised consolidation of the terrain such as column treatments like *jet grouting* or *deep mixing*.
The interventions can be based on the following provisions, or a combination of them.

Widening the foundation by way of stringcourses or reinforced plates. The intervention must be realised by taking care to connect the old and new foundations as a means of obtaining a monolithic whole which serves to diffuse tension in a homogeneous manner. To achieve this, r.c. beams and clamping, steel transverse elements with appropriate stiffness, post-tightened bars which guarantee the transmission of frictional forces and similar provisions can be utilised. This type of intervention also has beneficial effects for realising effective junctures between the masonry at the foundation level.
The insertion of small diameter, bored stakes (micro-stakes, root stakes) is an intervention which modifies the behaviour of the foundation in a significant manner, and therefore as a rule should be extended to the entire buildings and not limited to the instable portions. An appropriate connecting structure is always necessary between the stakes and the existing foundation (for example, reinforced stringcourses connected to the foundation according to the techniques described in the above section), unless the stakes are bored into the masonry with a perforation of sufficient depth to transfer loads caused by friction. In this case, the resistance of the existing structure must be verified in the mutated support conditions, allowing a cautionary hypothesis that all load agents will be transferred to the stakes.
Consolidation of the foundation terrain: The intervention methods may be chosen from a wide range of typologies, for example the injection of cement mixtures, resins, silicates, or other chemicals, or column treatments of *jet grouting* or *deep mixing*. Such interventions, in principal, should be avoided whenever the presence of archaeological substratum has been revealed.
During work, all interventions listed herein produce repercussions in the structure that can vary according to the type of intervention and terrain. These repercussions should be weighed accordingly, both by planning interventions to minimise them, as well as intervening on foundations prior to structural ones in elevation in order to perform any necessary repairs.
In situations where seismic activation is plausible due to phenomena of instability on slopes, the problem must be dealt with on the level of terrain and not only on that of the structural foundation.

## 6.4 Design procedures

### *6.4.1 Introduction*

Structures and their contents which undergo strengthening intervention projects for publicly owned historic buildings are governed by chapter XIII of the legislative decree, D.P.R. 554/1999 (*Regulations which govern projects for public works (Regolamento di attuazione della legge quadro in materia di lavori pubblici), no. 109 dated 11.2.1994*) and by Decreed Law no. 30 dated 22.1.2004, (*Regulations for public tenders for projects on cultural heritage (Disciplina degli appalti pubblici di lavori concernenti i beni culturali)*), besides the two publications of MiBAC (no. 42 dated 5.4.2002 and no. 20 dated 16.2.2004).
Even if the regulations deal specifically with public works, we retain that the general setting is adaptable to the private sector.
The D.P.R. 554/1999 (chapter XIII, section II) articulated restoration projects in three phases (preliminary, definitive, and executive projects) according to the same scheme used for general public works (D.P.R. 554/1999, chapter III, section II).
However, the elaboration of restoration projects is distinguished for various reasons. First of all, it is not necessary to produce all of the elaborate requirements of the law (chapter III, section II), but it is possible to demonstrate only documentation which is compatible with the specifics of the building subject to intervention (chapter XIII, section II, art. 213, paragraph 4). Second, it requires that the process of acquiring knowledge, analysis, and diagnosis of the building in question are part of an integral restoration project. It is assumed that knowledge of the building, an indispensable element to restoration projects, can never be exhaustive enough before beginning work on-site, and therefore must be elaborated in later phases, with an iterative procedure which aims to optimise the intervention. For this reason, for example, the executive project can be made in successive periods (within a timeframe outlined in the definitive project) and when necessary, can provide new in-depth insights to the survey and complete earlier research.
Although within the framework of the aforementioned legislation, the following paragraphs give specific documentation to attach to restoration projects as a means of documenting the evaluation process for seismic safety which is the object of these Guidelines.

### 6.4.2 *Preliminary project*

According to art. 214 of the Legislative Decree no. 554/1999 the project must show a framework for preliminary knowledge relative to the construction in question, and must synthetically illustrate the intervention methods which were chosen and will be further studied within the context of the definitive project.
As an end to creating a uniform framework for the acquisition of preliminary knowledge about the building, it is possible to refer to the forms illustrated in Attachment A and perform a Level 1 evaluation (LV1). All of the information gathered in this way will not yet be sufficient for the creation of the definitive project, but constitutes an instrument for obtaining an overall framework of the construction and identify the most critical areas. These areas must then be studied in greater depth through specific diagnostic surveys. Thus, the preliminary project must include in addition to the definition of the interventions, the planned survey projects which must be done during the definitive phase.
Nevertheless, the preliminary project must furnish a preliminary evaluation of the seismic safety of the building in its present state, which can be accomplished by way of LV1 tools.
In addition to what is required by law (D.P.R. 554/1999), the preliminary project must also contain the following documentation:

- Identification and knowledge of the construction through models proposed by the Ministry of Cultural Heritage and Activities (Attachment A);
- The program of diagnostic surveys to be accomplished during the definitive phase;
- The illustrative realisation of the construction in its present state, with descriptions of its seismic history and its seismic behaviour ascertained on a qualitative basis, and preliminary evaluations of seismic safety by way of simplified level 1 models (LV1).

### 6.4.3 *Definitive project*

In the phase of definitive design, detailed investigative surveys must be performed which define the confidence factor, evaluate the actual seismic safety as well as follow the strengthening intervention by way of LV2 or LV3 evaluation procedures.
Besides what is established in paragraph 1 of art. 215 of the D.P.R. 554/1999, the definitive project must contain an illustrative report of all evaluative processes performed, and justify their congruity to the level of seismic security after the designed seismic strengthening intervention. The report must:

- Define the motivations for the chosen reference level of seismic action on the site;
- Illustrate the results of the diagnostic analysis performed on the building for discerning the constructive materials, the characteristics of the materials, the interpretation of the instability present, and the identification of possible seismic damage mechanisms;
- Illustrate the mechanical model of the structure adopted for the seismic analysis, motivating the choice by the type of analysis performed and placing it in a framework according to what is outlined in LV2 or LV3 evaluations;
- Supply a final judgement of the effectiveness of the intervention by way of qualitative considerations and on the basis of a non-binding comparison between the structural capacity found in the calculation model with the reference level of seismic protection.

### 6.4.4 *Executive project*

The executive project, besides what is established in paragraph 1 of art. no. 216 of the D.P.R. 554/1999 should also:

- Outline the executive modalities of the operational techniques to be performed;
- Define any eventual further surveys to be realised on-site during the first phase of the work project;
- Indicate the controls to effectuate on-site, also with reference to the proper execution of the effectiveness of the intervention performed (in the case of interventions to cultural heritage, art. no. 187 of the D.P.R. 554/1999 which requires the obligatory inspections during the work).

This can also by given in later phases of the intervention, within the timeframe established in the definitive project. Only if necessary, new in-depth surveys may be needed based on previously obtained results.

Art. no. 219 of the D.P.R. 554/1999 sanctions the necessity of adjusting the executive project findings during restoration (which may have varied during work), on the basis of the results of these surveys and their findings after the worksite has been opened.

# 7 SUMMARY OF THE SEISMIC SAFETY EVALUATION PROCESS AND INTERVENTION DESIGN FOR SEISMIC IMPROVEMENTS

In the preceding chapters, indications for the evaluation of seismic safety has been provided for protected historic buildings to be performed according the requirements of the Code, of the NTC and of the Ordinance. This chapter simply summarises these, and does not add new information to what has already been indicated, although by no means should it be considered exhaustive.
The level of seismic protection can be differentiated by classifying the artefact according to three diverse categories of importance and three categories of use (point 2.4). For each class of importance or use, seismic actions have been defined by way of specific values regarding the probability of exceeding peak ground acceleration over a 50 years period in conditions of rigid terrain. In a similar manner, for the SLU, corresponding factors of importance $\gamma_I$ may be utilised which multiply horizontal and ground acceleration $a_g$. For the SLD other values for the probability of exceeding values in the next 50 years may be utilised, evidently higher than preceding ones, or the corresponding $\gamma_I$ factors. Average $\gamma_I$ values have been provided to be utilised when seismic danger studies are not available which permit the direct determination of acceleration having assigned the probability of exceeding values.
References for horizontal ground acceleration do not necessarily have to be added on the basis of seismic zoning of the territory, but may be derived from more accurate estimates of seismic danger (point 3.2).
For the evaluation of seismic capacity of constructions, the following were introduced and explained:

- Three evaluation levels (LV, point 5.3), corresponding to diverse seismic safety analysis needs:
  1) The evaluation of vulnerability of cultural heritage on a territorial scale;
  2) The design of seismic strengthening interventions for single elements of the construction;
  3) The design of seismic strengthening interventions which involve the behaviour of the entire building.
- A confidence factor ($F_C$, point 4.2), based on the grade of the depth of surveys performed on the building, considering implicit uncertainties of knowledge. Confidence factors are applied to mechanical parameters for materials or directly to the seismic safety evaluation, based on the calculation model employed.

Table 7.1 briefly summarises the existing relationships between the objectives of analysis, levels of evaluation, and calculation models.

Table 7.1 – Summary for the evaluation of seismic capacity.

| Seismic risk analysis of cultural heritage | | |
|---|---|---|
| Analysis objectives | Minimum evaluation level | Calculation Model |
| Evaluation on a territorial scale of the seismic safety index | LV1 | Simplified models (on a mechanical, statistical or qualitative basis) |
| The detailed ascertainment of the seismic safety of a single building | LV3 | Local thorough collapse mechanisms. Global models |
| Design of seismic strengthening interventions | | |
| Analysis Objectives | Minimum Evaluation level | Calculation model |
| Local interventions on limited areas of the building | LV2 | Local collapse mechanisms of single portions of the building |
| Interventions which involve the seismic | LV3 | Exhaustive local collapse |

| | | |
|---|---|---|
| functionality of the entire building | | mechanisms. Global models |

The comparison of actions and seismic capacity on single buildings can be performed by defining a seismic safety index $I_S$ (see paragraphs 2.1 and 5.3.1), which assumes a different aim in an analysis on a territorial scale or in a project for a seismic strengthening intervention.
In the first case, the seismic safety index is useful for a complex understanding of the level of seismic risk to the Italian cultural patrimony and to establish a list of priorities for planning preventative interventions.
In the case of designing seismic strengthening interventions, assuming that it is never necessary to proceed with an adjustment to the safety level required for new constructions, the values of the seismic safety index must not be intended as a parameter for compulsory verification ($I_S \geq 1$), but as an important quantitative element to consider along with others in a complex qualitative judgement that takes into consideration conservation criteria, the desire to protect the building from seismic damage, and safety requirements in relation to their fruition and function. All this must be described in an explanatory report of the solutions chosen for the project, especially in cases where the structural verification with calculation models is not completely satisfactory.