2. Seismic risk and safety assessment of the architectural heritage

The first part of this chapter reports a few concepts on seismic hazard which are relevant to the protection of cultural heritage, as well as a number of experiences learned from past earthquakes. In the second part, two questions are raised that are of great topical interest to the safety assessment of historical buildings, namely the viability of performance-based approaches, as proposed recently for application to the cultural heritage, and the role of recent monitoring technologies.

2.1 Seismic hazard and cultural heritage: geological, geomorphological and geotechnical aspects

The study of earthquakes goes back many centuries. There are written records of earthquake in China which dates 3000 years while in Europe to the last 2000 years. The occurrence of earthquakes on Earth is concentrated in specific regions, near geological active faults, as it is clearly seen from figure 2.1. It is quite interesting to point out that areas with relevant architectural heritage such as Europe, China or Middle East are active seismic zones.

From the study of geology, it has become evident that the source of earthquakes is the rapid movement of these active faults that cause massive release of elastic energy. When an earthquake occurs, seismic waves radiate away from the source and travel rapidly through the crust. The seismic waves can be distinguished in two main types: body waves and surface waves. Body waves can be further classified in primary waves (P-waves) that are compression longitudinal waves and in secondary waves (S-waves) that are transversal shear waves. P-waves are usually twice as fast as S-waves. Surface waves are generally slower than body waves, and due to their low frequency range, long duration and large amplitude they can be the most destructive for civil structures. They can be further classified as well in Rayleigh waves, also called ground roll, that are similar to those of waves on the surface of the water, and in Love waves (L-waves), that cause circular shearing in the ground [1].

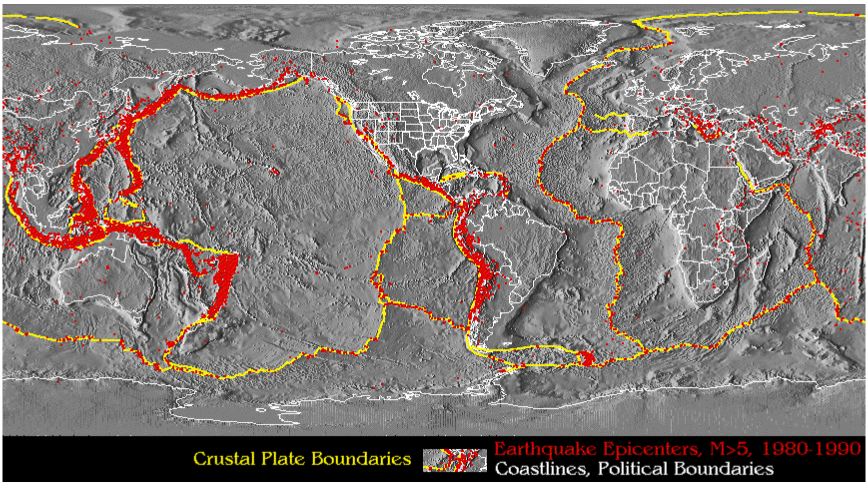

Figure 2.1 - Epicentres of earthquakes (1980-1990) [2].

2.1.1 *Local site effects and amplification*

The effects of the earthquake are directly related to the distance from the source. Anyway, whilst the previously defined seismic waves travel through the rock for most of the trip between the source of the earthquake and the surface, the final portion of that trip is through soft soil, which can greatly influence the nature of shaking at ground surface. The influence of local soil conditions has been recognised for many years. Since the 1920s, seismologists and geotechnical engineers have worked toward the development of quantitative methods for predicting the influence of local soil conditions on strong ground motion, usually classified in one-, two- or three-dimensional ground response techniques. In fact, the difference between free surface motion and bedrock motion, as defined by figure 2.2 can be great, due to amplification effects. Similarly to any mechanical system, the soil layers have several natural frequencies, and if the frequency content of the earthquake spectrum is close to one of these natural frequencies, resonance will occur [1].

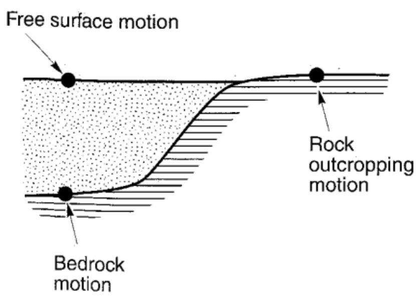

Figure 2.2 - Ground response nomenclature, vertical scale is exaggerated [1].

2.1.2 Soil-structure interaction

Until recently, the interaction occurring between structure and soil, during earthquake excitation has been often overlooked. Anyway, modern seismic standards (e.g. [3,4]) specify that this interaction cannot be neglected, especially in the case of massive buildings, as often are architectural heritage fabrics.

In order to illustrate the various effects of soil-structure interaction it is used an example taken from [1,5]. As shown by figure 2.3, two identical structures one founded on rock and the other embedded in soil are compared. For what concerns the structure on the rock, the horizontal motion can be applied directly at the base of the structure. The input acceleration will result in inertial loads constant over the height of the structure. Therefore, during the earthquake, an overturning moment and a transverse shear acting at the base will develop. Because rock is very stiff, these two stress resultants will not lead to any deformation at the base. Resuming, the seismic response of the structure depends only on the properties of the structure.

On the other hand, when dealing with the structure founded on soft soil, the motion of the base of the structure in point O has to take in account several effects, as shown in figure 2.3d-e. If there were no soil on top of the rock, such as in the case b, the motion of point C would hardly differ from the motion of the rock point A. It is noteworthy that in case c, the motion in the points D and E will differ from the motion in C, due to the presence of the soil layer. In general, the motion will result as amplified (but not always: it will depend on the frequency content), thus resulting in horizontal displacements increasing toward the surface of the soil. The insertion of a rigid base into the site (case d) will result in a modification of the motion. In fact, the rigid base will experience some average horizontal displacement and a rocking component. This rigid-body motion will result in accelerations (leading to inertia loads) which will vary over the height of the structure. This will be the result of the so-called kinematic-interaction part of the analysis. Finally, the inertial loads applied to the structure will lead to an overturning moment and transverse shear acting at point O (figure 2.3e). These will cause deformations in the soil and thus modify the motion at the base.

In this latter case, because of the amplification of the site, the translational component will, in many cases, be larger than the motion and, furthermore, a rocking component will arise. This introduced rocking component will affect the response significantly, especially in the case of tall structures. Another aspect to take in account is the radiation of energy propagating waves away from the structure: this will result in an increase of the damping of the final dynamic system.

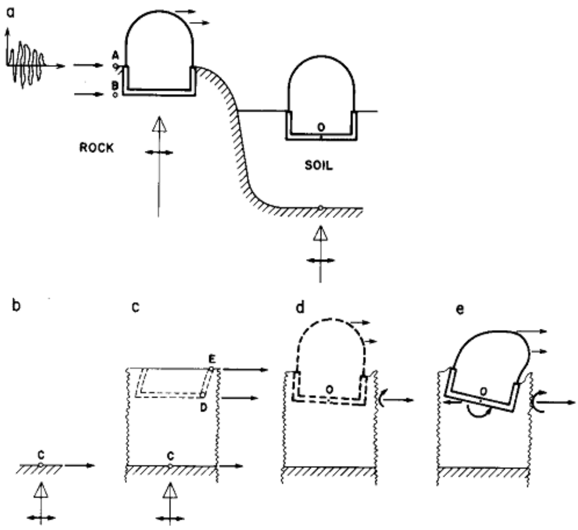

Figure 2.3 - Seismic response of a structure founded on rock and soil: (a) Sites; (b) outcropping rock; (c) free field; (d) kinematic interaction; (e) inertial interaction [5].

A classical representation of the problem exposed hereinbefore, is the one proposed in [5], by using a simple SDOF system, mounted on a rigid, massless foundation supported on an elastic soil deposit. The stiffness and damping characteristics of the compliant soil-foundation system can be represented by the translational and rotational springs and dashpot shown in figure 2.4.

Neglecting the analytical solution of the system, which can be found in both references [1] and [5]; it is interesting to mention the qualitative behaviour of this system, which reflects the considerations of above.

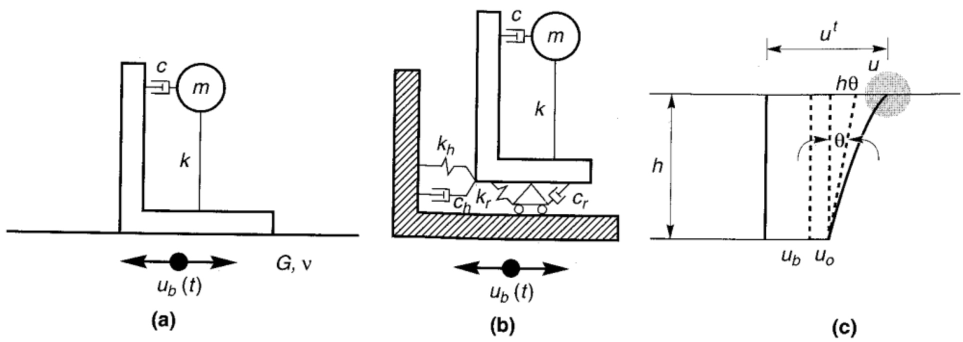

Figure 2.4 - Compliant base model with one dynamic degree of freedom: (a) SDOF system on elastic soil deposit, (b) idealized discrete system in which compliance of base is represented by translational and rotational springs and dashpots, (c) components of motion of base and mass [1].

In fact, studying the frequencies relative to the various springs represented in figure 2.4 (where k represents the stiffness of the structure, k_h the horizontal stiffness of the soil and k_r the stiffness of the rocking movement), it is findable that the natural frequency of the whole equivalent system is always lower than the one of the fixed-base structure. Therefore, an important effect of soil-structure interaction is to reduce the natural frequency of the soil-structure system to a value lower than that of the structure under fixed-base conditions. Moreover, if one defines a stiffness ratio as a ratio between the stiffness of the structure respect to the stiffness of the soil, it is possible to find that the higher the stiffness ratio the lower the ratio between equivalent frequency and natural frequency of the structure. Again, as stated before, it is confirmed that when the stiffness of the soil is large relative to the stiffness of the structure the soil-structure interaction is negligible.

In more complex systems, such as in the case of architectural heritage structures, one should usually resort to computer software, among which most use the finite element method for the structure and the boundary element method for the soil and allow for resolution of complicated problems of soil-structure interaction (see for instance [6,7]).

2.1.3 Liquefaction

Liquefaction is the geotechnical dynamic phenomenon that, if occurs, may produce the most tragic consequences for buildings. The phenomenon has been largely studied in the last 50 years, especially after the Great Alaskan earthquake and the Niigata earthquake, in Japan, both occurred in the 1964. These two earthquakes caused a lot of spectacular examples of liquefaction, causing slope failures, flotation of buried structures or building foundation failures (see, for instance, figure 2.5).

Figure 2.5 - Structures damaged in the Marina District of San Francisco following the Loma Prieta earthquake. The first story of this three-story building was damaged because of liquefaction; the second story collapsed. What is seen is the third story (Credit: Plafker, G, Figure 24B, U.S. Geological Survey Circular 1045).

Liquefaction is a term generally referred to the tendency of saturated cohesionless soils subject to rapid loading to decrease its effective stresses due to the sudden increase of interstitial water pressure, for a thorough analysis on the . Liquefaction is a result of processes that can be classified in two main groups: flow liquefaction and cyclic mobility [8].

Flow liquefaction produces the most dramatic effect, instabilities usually called flow failures. It occurs when the shear stress required for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state [1]. Flow liquefaction is characterised by a sudden and unexpected type of failure, a great speed of development and a large distance over which the liquefied materials often move. A classic example of flow slide failure is shown in figure 2.6, the collapse of the Sheffield dam as a tragic consequence of the Santa Barbara earthquake (1925).

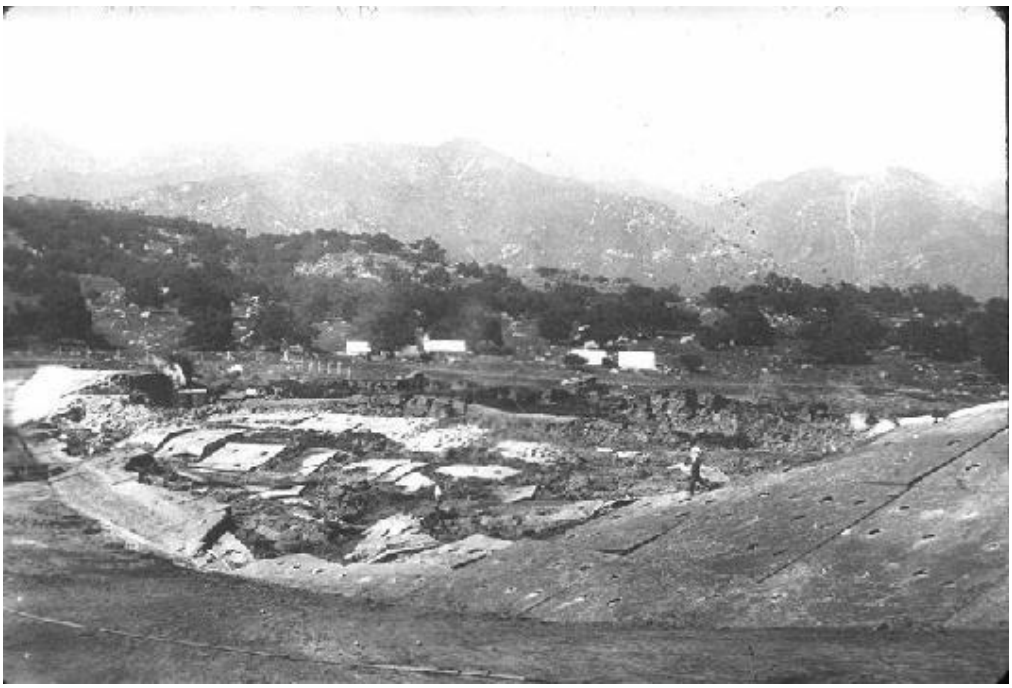

Figure 2.6 - Collapse of the Sheffield dam after the 1925 earthquake, located at the base of the Santa Ynez Mountains at the northern end of Santa Barbara (USA).

Cyclic mobility is another phenomenon that can produce unacceptably large permanent deformations during a shear loading process, such as an earthquake. In contrast to flow liquefaction, cyclic mobility occurs when the static shear stress is less than the shear strength of the liquefied soil. The deformations develop incrementally during an earthquake if cyclic mobility occurs. For further reading on the topic see [9,10].

Furthermore, a special case of cyclic mobility is level-ground liquefaction. These types of failures are characterised by the upward flow of water that occurs when the excess of pore pressure dissipates. Depending on the length of time require reaching hydraulic equilibrium, level-ground liquefaction failure may occur well after ground shaking has ceased. Excessive vertical settlement and consequent flooding of low-lying land (occurred also in the Tokyo streets during the recent Tohoku earthquake) and the developments of sand boils are characteristic of this type of failure.

2.2 Seismic vulnerability of architectural heritage: experiences learned from past earthquakes

In the history of human being earthquakes have always been among the most important sources of devastation and there are often written records of the most tragic earthquakes. Most of the buildings making part of the architectural heritage had experienced an earthquake in their history. In fact, important structures such as religious buildings, palaces or castles have very long service life, and they may still be used nowadays. As said, the greatest part of the architectural heritage is concentrated in the Mediterranean area and in the Far East: both the zones are characterised by elevated seismic activity. Observing these two regions of the world it is easily possible to distinguish two different traditions of construction: stone and masonry for the western architecture and wood for eastern architecture. Typically, the Mediterranean area is characterised by stone (think about Egyptian pyramids or the Greek Parthenon) and masonry structures (think about roman architecture or gothic cathedrals) both for religious and civil buildings. These types of structures are massive, really stiff and can be classified as “strong”. Anyway, when subject to earthquakes massive structures require much greater resistance than an equivalent flexible and light structure. Another point to make is that masonry “remembers” its load history and, therefore, may accumulate damage. That has been said, well-constructed and realised masonry buildings can withstand strong earthquakes as witnessed by the high number of masonry buildings survived in seismic areas.

On the other hand, wood buildings do not have great masses and are typically showing good resistance with respect to their stiffness. But, usually cannot guarantee the same strength as masonry or stone buildings allowing for the realisation of structures of the same size.

2.2.1 Earthquakes and architectural heritage in Europe and Middle East

Typically, among the architectural heritage, masonry and stone buildings are the most subject to the tragic effects of earthquakes due to their relatively high masses which greatly increase the effects of seismic actions. Moreover, even low intensities earthquakes can represent a hazard for masonry historic buildings: in fact building which life is counted in centuries may accumulate damage and the structure can be significantly weakened after many weak seismic events.

Earthquakes shaped the world where we live and it is possible to witness their effects on notorious heritage buildings. For instance, the Coliseum in Rome has been notably damaged through its bi-millenary life by earthquakes, mostly during the 443, 508 and 1703 earthquakes [11]. Historic analysis allows reconstructing the seismic damages which an architectural building underwent; it is an example the deterioration of the Coliseum due to earthquakes and anthropic actions shown in figure 2.7.

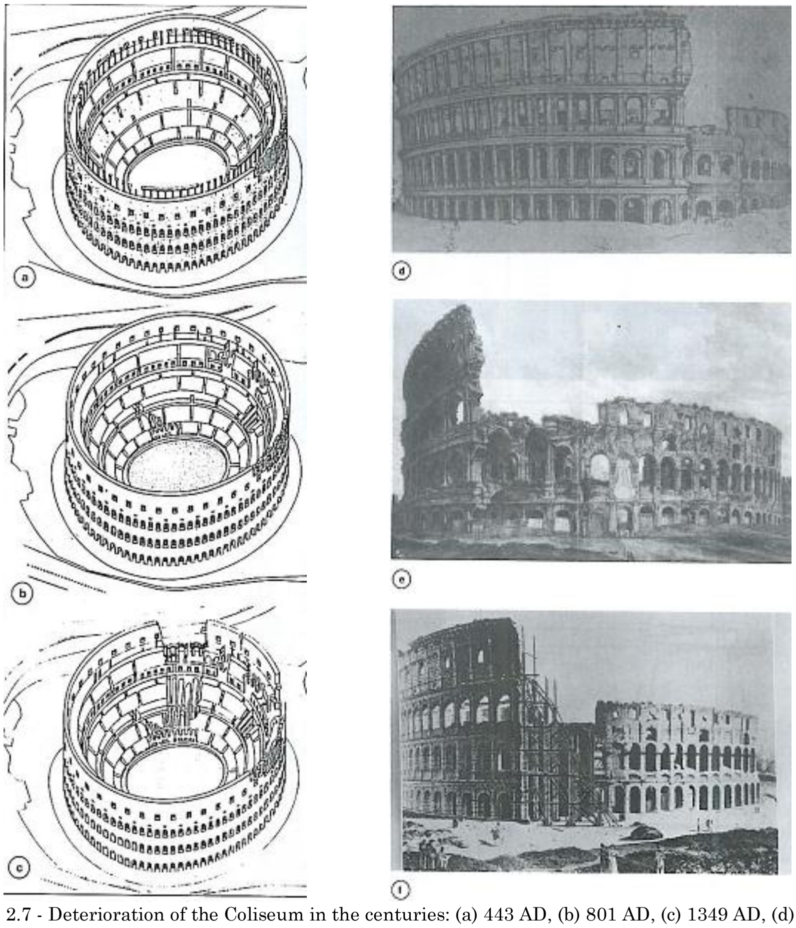

Figure 2.7 - Deterioration of the Coliseum in the centuries: (a) 443 AD, (b) 801 AD, (c) 1349 AD, (d) XV century, (e) after 1703 earthquake, (f) realisation of Valadier abutment.

It is quite straightforward that not all the architectural heritage that was subjected to earthquakes survived to our days. In fact the history of human kind contains numerous examples of notorious lost structures such as the Lighthouse of Alexandria, severely damaged by an earthquake in 956 AD and definitely destroyed by two earthquakes in 1303 and 1323 or the Mausoleum of Halicarnassus, probably damaged and destroyed by earthquakes (completely destroyed by 1404 AD when only the foundations were still recognizable). Another case, well-known in Italy, is the complete destruction of the church of Santissima Annunziata in Messina occurred during the 1908 earthquake, the most destructive European earthquake of the last century, which was a masterpiece of the baroque architect Guarino Guarini (see figure 2.8).

Figure 2.8 - Church of Santissima Annunziata, lost during the 1908 Messina earthquake.

In the last two centuries the number of documents available has increased exponentially due to new technologies (photography, filming …) this allowing classifying in an easier fashion the types of damage occurred to the structures subject to earthquakes. In the last decades with the new media technology it has been possible to document earthquakes almost instantaneously. Probably, one of the most famous examples of earthquake caused damage to architectural heritage in the last years is the partial collapse of one of the vaults of the basilica of San Francesco in Assisi (Italy) during the 1997 earthquake. The collapse destroyed the frescos on the vaults as it is shown by figure 2.9.

Figure 2.9 - Collapse of the vault of the San Francesco Basilica in Assisi after the 1997 earthquake.

This collapse had an impressive impact on the public opinion due to the availability of a video of the collapse, and due to the artistic importance of the Basilica, which is a masterpiece of the Italian gothic. The same earthquake damaged also others important historic buildings such as the Civic Tower in Foligno (see figure 2.10).

Figure 2.10 - Damage occurred to the Civic Tower of the city hall in Foligno after the 1997 earthquake.

More recently, in 2009, another earthquake had a global impact with what concern the protection of cultural heritage sites from seismic events. L'Aquila, a historic city located in centre Italy, has been hit by an earthquake and most of its numerous churches and palaces have been heavily damaged. The list of the damaged buildings is quite long. Among the others, the basilica of Santa Maria di Collemaggio and the church of Anime Sante partially collapsed. Figure 2.11 shows that drum-dome systems, in particular, can be critical in masonry churches.

Figure 2.11 - Basilica of Santa Maria di Collemaggio (left-side), Church of Anime Sante (right-side).

Other two Mediterranean countries that have to deal with strong earthquakes are Greece and Turkey. For instance, in Greece, the Katholikon of Daphni (an UNESCO World Heritage Site) suffered throughout its history a large number of earthquakes that caused many structural problems and damage to it. The last one was in 1999 and severely damaged the monastery, forcing an emergency intervention as reported in figure 2.12 [12].

Figure 2.12 - Emergency intervention after the 1999 earthquake at the monastery of Daphni.

Another significant example of monumental structure which has been subjected to a turbulent seismic history is the magnificent church of Hagia Sophia in Istanbul, nowadays a mosque. Hagia Sophia, built in the V century, has been damaged by earthquakes through all its history. The original dome collapsed for the first time in 558 AD, due to an earthquake. Other earthquakes, in 869 and 968 AD, damaged the western part of the building. The structure was then subject to numerous sources of anthropic damage (such as in the sacks of Constantinople in 1204 and 1453) and other minor earthquakes. Nowadays the structure is one of the most monitored in the world as witnessed by the numerous studies executed in the last decades (see for instance Aoki *et al* [13]).

In the case of weak types of masonry, such as adobe, the effects of an earthquake can be even more devastating, as in the case of the Bam earthquake in 2003. The Bam citadel, which is a UNESCO World Heritage Site located in Iran, has been completely destroyed by an earthquake as it is shown in figure 2.13.

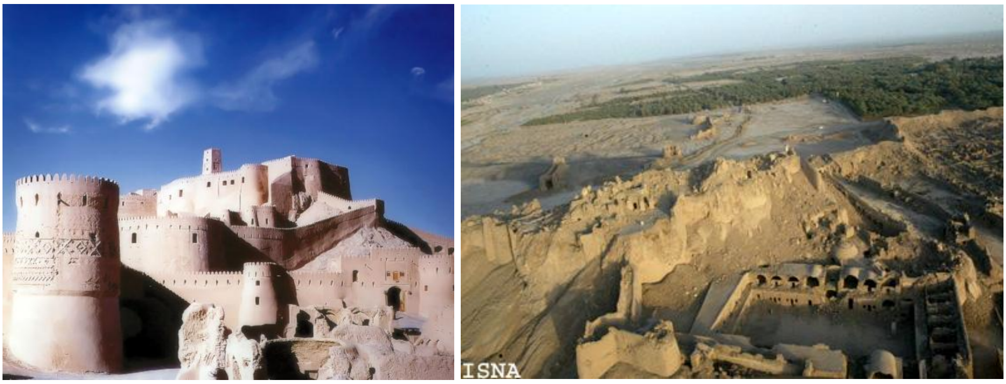

Figure 2.13 - The citadel of Bam before and after the Bam earthquake of 2003.

India falls again in the area of masonry and stone structures. The zone is often subject to strong earthquakes such as in 2001 in the region of Gujarat. The earthquake collapsed the oldest and most important of the Chhatris, the 18th Century Rao Lakha Chhatri (see figure 2.14a). This was an elaborate open stone pavilion, with a central dome and twelve smaller domes lining its stepped façade. The earthquake caused it to completely collapse into a formless pile, even though it had survived the great 1819 earthquake. During the British Raj period, Maharajas constructed new beautiful palaces on the edges of the cities and towns, called Darbargarh. Most of these palaces are considered to be heritage sites and have been damaged during the previously mentioned earthquake, as it is shown in figure 2.14b.


Figure 2.14 - Chhatris showing collapsed Raolakha Chhatris on right (left side). Pragmahal, Darbargarh of the Maharaja of Kutch (right side).

2.2.2 *Earthquakes and architectural heritage in the Far East*

In the Far East, earthquakes are relatively stronger than in the Mediterranean area, and ancient populations had to deal with it. The first answer, as it is possible to see nowadays, was to use for constructions a widely available, light and relatively stiff material: the wood. In fact, wood has one of the best stiff to mass ratios, comparable to the one of steel. In the case of a flexible structure the global behaviour is generally different than the behaviour of a stiff structure: whilst a stiff structure "resists" an earthquake, a flexible structure "follows" it. In order to emphasise this wood behaviour, ancient oriental buildings were realised without the use of nails or screw but only realising inlaid joints with wood elements.

The second answer to earthquake was to use the concept of floating foundations, usually called "overall floating rafts foundations". In this case the building just "floats" the earthquake waves, reducing the effect of seismic actions on the upper part of the building.

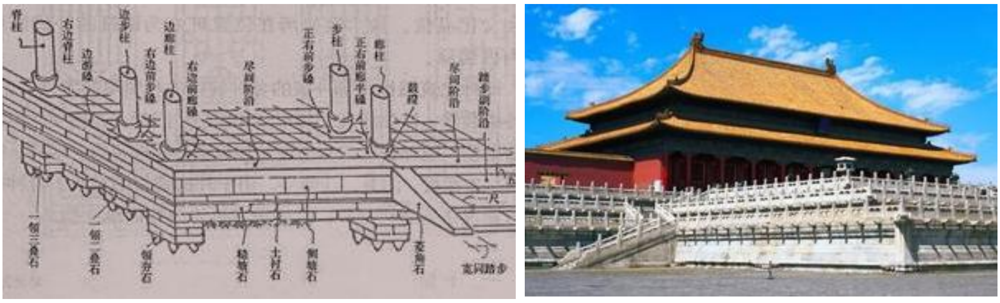

Figure 2.15 - The concept of floating foundations (left) and the Forbidden City in Beijing (right).

Last, but not least, in the Chinese architecture there is the "Dou-Gong" which is a unique structural element of interlocking wooden brackets (Figure 2.16). The function of Dou-Gong is to provide increased support for the weight of the horizontal beams that span the vertical columns or pillars by transferring the weight on horizontal beams over a larger area to the vertical columns. Adding multiple sets of interlocking brackets or Dou-Gong reduces the amount of strain on the horizontal beams when transferring their weight to a column. The elements have experimentally shown good earthquake capabilities due to its ductility and flexibility [14].

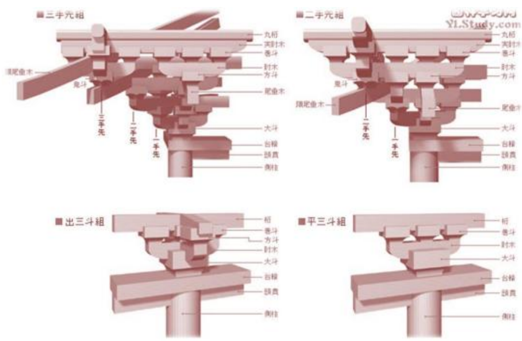

Figure 2.16 - The Dou Gong system

This having been said, also wood structures can suffer heavy damage from earthquakes. The World Heritage Site of Du Jiangyan, a renowned scenic spot in Qing Mountain, in China, has been damaged by the Sicuan earthquake in May 2008. The site has been heavily damaged, but the structure of the temple did not collapse (see figure 2.17).

Figure 2.17 - Damage occurred to the Er Wang temple after the Sicuan earthquake in 2008 (Magnitude 7.9).

This was not the case of an historic earthquake, occurred in 1934 in Bhaktapur, in the region of Nepal which is nowadays a World Heritage Site. In this situation it is interesting to observe the behaviour of a traditional temple with timber frame. Its mass and rigidity are distributed equally and symmetrically (Figure 2.18). Therefore, the point of resultant forces of earthquake action in earthquake almost meet the point of resultant forces of resistance, thus avoiding or reduce the torsion of the building, helpful for shock resistance. An analogous temple, but built with a masonry structure completely collapsed during the same earthquake (Figure 2.19).

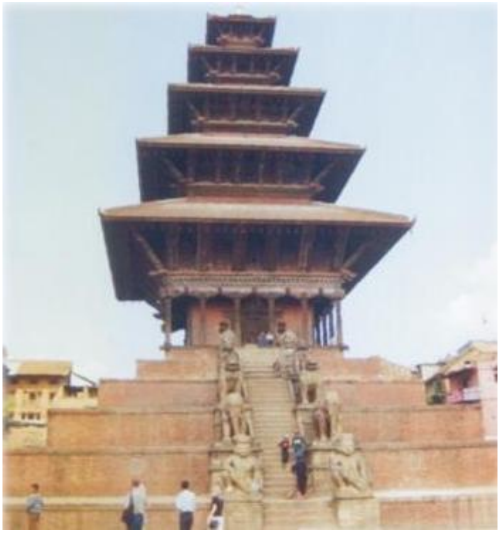

Figure 2.18 - The Nytapola temple (Bhaktapur, Nepal) before and after the 1934 earthquake.

Figure 2.19 - Degu Taleju Temple. Top: This large building with huge wall thickness collapsed completely. Bottom: Contemporary view after reconstruction.

2.3 Performance-based approaches in structural engineering problems and codes

Performance based approach is defined by Gibson [15] as "the practice of thinking and working in terms of ends rather than means. It is concerned with what a building or a building product is required to do, and not with prescribing how it is to be constructed". Therefore the approach targets on the required performance for the building.

The use of performance based design is nowadays a standard in most of the national structural engineering codes across the world. Anyway, especially in developed countries, new constructions account for only a small percentage of the total building inventory. Applying the codes developed for new buildings to existing buildings may be misleading and inappropriate. In fact, in the case of existing buildings the situation is extremely more complicated due to the high individualities and the huge differences with respect to the age of construction, the condition of the existing building and also with respect to the social environment [16]. For example there may be substantial differences between countries on the notion of the rights of a homeowner and regarding societal preferences for preservation as opposed to replacement of old buildings. It is also interesting to notice that whilst for some national codes a renovation, extension or change of use of a portion of an existing building will require the entire building to be upgraded, in other countries only the portion of the building affected by this transformation will be covered by current regulations for new buildings and the rest of the building would not need to be upgraded.

2.3.1 Performance-based approach for existing buildings

Recent structural engineering standards, both national and international [17,18,19,20,21], introduced the performance based approach for the rehabilitation of existing buildings. In such a case, it is usually recognised that it is impossible to apply the requirements for new constructions and, in some circumstances, it is usually accepted that the performance expected when upgrading existing buildings is less pressing than for new constructions.

Generally speaking, these codes define a performance level (or a performance range) which, in the case of seismic design, is the desired post-earthquake condition of the building (or the band of conditions) [22]. It is worth noticing that often a structural performance level and a non-structural performance level are introduced, selected among a wide set of damage states that buildings can undergo after a seismic event. For instance, it is interesting to analyse the cases of the ASCE 41-06 [17] and of Eurocode 8 [18,19], in order to see how performance levels can be addressed.

ASCE 41-06 is the technical code which is in force in the United States. The code has superseded the FEMA 356 [20], which firstly introduced the performance based approach for existing buildings. In first place, the ASCE 41-06 defines several levels of target building performance levels, divided in structural performance and non-structural performance. The code distinguishes six levels of structural performance, defined as follows:

- Immediate occupancy structural performance level (S-1): in this case only very limited structural damage has occurred as consequence of earthquake. Basically, the building retains its pre-earthquake strength and stiffness.
- Damage control structural performance range (S-2): is a continuous damage range which goes from S-1 to S-3, this range may be desirable to minimize repair time and operation interruption, as a partial means of protecting valuable equipment and contents, or to preserve important historic features when the cost of design for immediate occupancy is excessive.
- Life safety structural performance (S-3): means that the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains.
- Limited safety structural performance range (S-4): is a continuous damage range which goes from S-3 to S-5.
- Collapse prevention structural performance level (S-5): it is the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse.
- Structural performance not considered (S-6): in this level the structural performance is not considered. This level will be considered in the case of a rehabilitation program of certain non-structural vulnerabilities (e.g. bracing parapets, anchoring hazardous materials…), without addressing the performance of the structure itself.

On the other hand, dealing with non-structural performance levels, the ASCE 41-06 standard addresses five performance levels:

- Operational non-structural performance level (N-A): it is defined as the post-earthquake damage state in which the non-structural components are able to support the pre-earthquake functions present in the building.
- Immediate occupancy non-structural performance level (N-B): it is defined as the post-earthquake damage state in which non-structural components are damaged but building access and life safety systems including doors, stairways, elevators,

emergency lighting, fire alarms, and fire suppression systems generally remain available and operable, provided that power is available.

- Life safety non-structural performance level (N-C): it is defined as the post-earthquake damage state in which non-structural components are damaged but the damage is not life-threatening.
- Hazards reduced non-structural performance level (N-D): it is defined as the post-earthquake damage state in which non-structural components are damaged and could potentially create falling hazards, but high-hazard non-structural components are secured to prevent falling into areas of public assembly. Preservation of egress, protection of fire suppression systems and similar life-safety issues are not addressed in this non-structural performance level.
- Non-structural performance not considered (N-E): a building rehabilitation that does not address non-structural components shall be classified as N-E.

The following step is the definition of the rehabilitation objectives. A rehabilitation objective is fulfilled when a performance level is verified with the respective seismic hazard level (defined by the norm as BSE-1 and BSE-2). Seismic hazards can be addressed both in a probabilistic or deterministic fashion: probabilistic hazards are defined in terms of the probability that more severe demands will be experienced (probability of exceedance) in a 50-year period. Deterministic demands are defined within a level of confidence in terms of a specific magnitude event on a particular major active fault. Consequently, the ASCE 41-06 defines three different rehabilitation objective levels:

- Basic safety objective (BSO): the buildings meeting the BSO are expected to experience little damage (S-3) from relatively frequent, moderate earthquakes (BSE-1), but significantly more damage and potential economic loss (S-5) from the most severe and infrequent earthquakes (BSE-2).
- Enhanced rehabilitation objectives: rehabilitation that provides building performance exceeding that of the BSO is termed an Enhanced Objective. Enhanced Rehabilitation Objectives shall be achieved using one or a combination of the following two methods:
  1. By designing for target Building Performance Levels that exceed those of the BSO at either the BSE-1 or BSE-2 hazard levels, or both.
  2. By designing for the target Building Performance Levels of the BSO using an Earthquake Hazard Level that exceeds either the BSE-1 or BSE-2 hazard levels, or both.
- Limited rehabilitation objectives: Rehabilitation that provides building performance less than that of the BSO is termed a Limited Objective. Limited rehabilitation objectives shall be achieved using reduced rehabilitation or partial rehabilitation, and shall comply with the following conditions:
  3. The rehabilitation measures shall not result in a reduction in the performance level of the existing building;
  4. The rehabilitation measures shall not create a new structural irregularity or make an existing structural irregularity more severe;
  5. The rehabilitation measures shall not result in an increase in the seismic forces to any component that is deficient in capacity to resist these forces;

6. All new or rehabilitated structural components and elements shall be detailed and connected to the existing structure in compliance with the requirements of this standard.

Table 2.1 shows several common target performance levels for building in a schematic way, and how they are linked with structural and non-structural performance levels.

Expected Post-Earthquake Damage State	
Operational (1-A) Backup utility services maintain functions; very little damage. (S1+NA)	higher performance less loss
Immediate Occupancy (1-B) The building remains safe to occupy; any repairs are minor. (S1+NB)	
Life Safety (3-C) Structure remains stable and has significant reserve capacity; hazardous non-structural damage is controlled. (S3+NC)	
Collapse Prevention (5-E) The building remains standing, but only barely; any other damage or loss is acceptable. (S5+NE)	lower performance more loss

Table 2.1 - Target Building Performance Levels and Ranges

Last but not least, it is worth reviewing briefly the acceptance criteria imposed by ASCE 41-06. The code distinguishes among primary and secondary structural elements and components, and further distinguishes actions as deformation-controlled (ductile) and force-controlled (non-ductile). The concept of primary and secondary elements permits the engineer to differentiate between the performances required of elements that are critical to the ability of the building to resist collapse and of those that are not. For a given performance level, acceptance criteria for primary elements and components will typically be more restrictive than those for secondary elements and components.

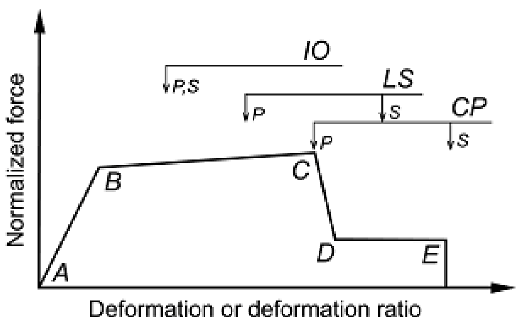

Figure 2.20 - Relation between normalised force and deformation at different building performance levels.

Figure 2.20 shows the generalised force versus deformation curve acceptance criteria for deformation or deformation ratios for primary members (P) and secondary members (S) corresponding to the target Building Performance Levels of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO). The graph can be distinguished in four response zones:

- linear response is depicted between initial point A and an effective yield point B;
- the slope from point B to point C is typically a small percentage (0%-10%) of the elastic slope, and is included to represent phenomena such as strain hardening;
- point C has an ordinate that represents the strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line CD);
- beyond point D, the component responds with substantially reduced strength to point E; at deformations greater than point E, the component strength is essentially zero.

The Eurocode 8 is a supranational technical code and is divided in various parts. The first part, entitled as "General rules, seismic actions and rules for buildings" defines the target performance levels for buildings as:

- No-collapse requirement: the structure shall be designed and constructed to withstand the design seismic action defined without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action may be expressed in terms of: a) the reference seismic action associated with a reference probability of exceedance, P_{NCR}, in 50 years or a reference return period, T_{NCR}, and b) the importance factor γ_I which relates to the consequences of a structural failure, and multiplies the reference seismic action.
- Damage limitation requirement: the structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The seismic action to be taken into account for the "damage limitation requirement" has a probability of exceedance, P_{DLR}, in 10 years and a return period, T_{DLR}.

When dealing with existing structures one must refer to part 1-3 of the Eurocode 8 [19]: "Assessment and retrofitting of buildings". In this case, Eurocode 8 introduces three different limit states as performance criteria. The three limit states have to be characterised as follows:

- Limit state of Near Collapse (NC): the structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity. It refers to a return period of 2475 years (occurrence probability of 2% in 50 years).
- Limit state of Significant Damage (SD): the structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity.

The structure is likely to be uneconomic to repair. It refers to a return period of 475 years (occurrence probability of 10% in 50 years).

- Limit state of Damage Limitation (DL): the structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures. It refers to a return period of 225 years (occurrence probability of 20% in 50 years).

When dealing with the assessment of structural elements Eurocode distinguish in “ductile” and “brittle” elements, whilst the first one have to be assessed in terms of deformations, the latter has to be assessed in terms of strength.

In the particular case of masonry buildings, which are of particular interest for this thesis:

- Limit state of Near Collapse (NC): global capacity at the Near Collapse limit state may be taken equal to the ultimate displacement capacity taken as the roof displacement at which total lateral resistance (base shear) has dropped below 80% of the peak resistance of the structure, due to progressive damage and failure of lateral load resisting elements.
- Limit state of Significant Damage (SD): global capacity at the Significant Damage limit state may be taken equal to ¾ of the ultimate displacement capacity (NC).
- Limit state of Damage Limitation (DL): the capacity for global assessment is defined as the yield point (yield force and yield displacement) of the idealized elasto-perfectly plastic force-displacement relationship of the equivalent Single-Degree-of-Freedom system.

2.3.2 *Towards a performance-based assessment of historical buildings?*

In light of what it has been shown in the previous sections, the extension of the performance based criteria to architectural heritage buildings seems to be a logical consequence. The already cited codes for existing buildings do not treat the peculiar case of historic buildings, or they are just mentioned.

A technical code that is worth mentioning is the Spanish code for the architectural restoration [23], currently under development, which introduces the performance based approach for structural assessment of historic buildings. Anyway, this code has been developed on the basis of the most state-of-the-art document, the recently introduced Italian guidelines for the reduction of seismic risk in cultural heritage buildings [4], where the performance-based approach is introduced for cultural heritage. The Guidelines are one of the few codes that deal with the seismic assessment of the architectural heritage. Following the Italian building code [21], the concept of Limit State has been introduced for historic buildings (see §2.3.1). In the case in point, the Directive proposes three different limit states:

- *Ultimate limit state* (SLU): the structure is severely damaged but still retains some residual strength with respect to horizontal actions and its full strength and capacity to carry vertical loads. It refers to an earthquake with an occurrence probability of 10% in 50 years.

- *Damage limit state* (SLD): the structure is slightly damaged but it can still undergo its normal use. It refers to an earthquake with an occurrence probability of 50% in 50 years.
- *Artistic limit state* (SLA): it is a new concept introduced by the Directive. In this case the artistic features of the structure (frescos, plasters...) are required to be undamaged or slightly damaged (but still recoverable). The reference earthquake can usually be the same of the Damage limit state.

The Guidelines [4] highlighted also another aspect to be taken into account for cultural heritage structures: the category of relevance of the building. In fact historic structures can require particularly high level of seismic protection, especially if they have a pivotal role as cultural heritage. On the other hand, the first version of the Directive PCM in 2008 [3] had set three different relevance levels for historic structures, namely: limited, average and high. On the other hand, the seismic action had to be defined not only by the relevance level in the artistic sense, but also by their usage category. Therefore three levels were defined also in this case: occasional, frequent, very frequent.

The European research project PERPETUATE [24] concerns precisely the application of the performance-based approach to earthquake protection of cultural heritage in European and Mediterranean countries. This project, leaded by the University of Genova (Sergio Lagomarsino), intends to develop European Guidelines for the evaluation and mitigation of seismic risk to cultural heritage assets, with innovative techniques for the seismic strengthening of historical buildings and the preservation of unmovable artworks. The project is approaching the problem from two points of view: for architectonic assets (historic buildings; macroelements, which are architectonic elements that may be analysed independently from the rest of the building) and for artistic assets (frescos, stucco-works, statues, pinnacles, battlements, banisters, balconies …). The problem is analysed by using two different scales: assessment of a single cultural heritage and assessment at the territorial scale with simplified vulnerability and risk analysis.

In one of the numerous deliverables of the project [25,26], some criteria for the choice of the target performance levels for the seismic retrofit of historic buildings are proposed in an even more detailed way than into the Directive. Especially for what concerns life safety and the safety of the artistic assets the Directive provides few details. Consequently, three levels of performance are proposed [25]:

- *Use and human life*: as well as for other type of buildings, the possibility of an immediate occupancy after an earthquake and the preservation of human life may be important for a cultural heritage asset;
- *Building conservation*: the preservation from building damage is not related, as for ordinary buildings, to the costs of repair or rebuilding but to the possibility of restoration, due to the intangible value of a cultural heritage asset;
- *Artistic assets conservation*: in many cases, irremediable damage to artistic assets can occur also in the case of moderate damage to structural elements; thus it is necessary to define specific target performance levels.

The target performance levels may be also associated to damage levels, which are usually defined by a typical push-over curve (a curve obtained by a non-linear static analysis and representing the top displacement versus the shear force). The curve usually shows a typical sequence of four phases:

- *Slight damage*: a progressive reduction of global stiffness (with the increase of local damage in structural elements) is observed; the slight damage is defined in this first branch of the capacity curve.
- *Moderate damage*: The global maximum strength is attained when a significant number of structural elements reach their maximum strength.
- *Heavy damage*: A ductile post-peak behaviour (thanks to the ductility of structural elements) follows, with limited softening; the heavy damage level may be defined at the end of this branch.
- *Complete damage*: A brittle behaviour or a remarkable softening phase follows, until collapse (due to the progressive collapse of elements at local scale).

The PERPETUATE proposal is to define, by using some of these damage levels, more specific levels to be achieved for the target performance. Only one of these levels will be considered the main and another has to be verified in specific cases; the remaining ones may be considered satisfied once the main and the secondary are verified. These levels are the following:

Use and human life

- *Operational*: the building is expected to sustain minimal or no damage. It is suitable for its normal occupancy and use, although possibly in a slightly impaired mode.
- *Immediate occupancy* (secondary): the building maintains its overall functionality, even if limited damage occurred or some parts may be not immediately usable. This performance level is important in those buildings in which relevant public functions (public offices, hospitals, schools …) or important cultural activities (museums …) are carried out (usability of cultural assets).
- *Life safety* (main): the building retains its structural integrity and a residual load bearing capacity after seismic events. Human life is preserved. Damage is very high but there is a low risk of local collapses, affecting the safety of people.

Building conservation

- *Damage limitation*: the building retains its load bearing capacity, but suffers limited damage which may require localized restoration interventions. A moderate damage is acceptable for cultural heritage assets, due to intrinsic fragility of masonry buildings and the possibility of repair (which is always preferable against over strengthening to prevent any damage).
- *Collapse prevention* (main): the building retains its overall structural integrity and a residual load bearing capacity. It can be restored, even if partial reconstructions may be required. The building may be conserved over time. In standard codes, the concept of reparability is often adopted. Usually, this performance level is associated with conditions of damage for which the costs of repair would be disproportionately high in comparison with the costs of the structure itself. In the case of cultural heritage assets, this concept may not be applied, due to the invaluable nature of the asset itself.
- *Ruins* (secondary): the building loses its structural integrity and may be conserved only as a ruin. In case of cultural heritage assets, the loss of structural integrity should not lead to the demolition of the building. Even if the building has lost its integrity and is no more usable, it may be conserved as a ruin.

Artistic assets conservation

- *Integrity*: loss of functionality of equipment and facilities aimed to the preservation of cultural heritage assets, against humidity and other decay factors.
- *Low damage* (main): artistic assets attached or contained in the building are damaged, but they can still be restored. The restorability of the asset should be mainly evaluated on the basis of technical feasibility.
- *Loss prevention* (secondary): artistic assets attached or contained in the building collapse; due to the destructive damage suffered, they can be restored only partially.

For what concerns the definition of the project earthquake and the seismic hazard, different return times can be used, varying from 50 to 2475 years, related to the different damage levels and performance criteria. The whole classification of damage levels, return periods and performance criteria is resumed in table 2.2.

Damage level	**Use and Human Life**	**Architectonic assets**	**Artistic Assets**	**Return period T_R**
Complete	-	Ruins	-	T_R=2475
Heavy	Life Safety	Collapse Prevention	Loss prevention	T_R=475
Moderate	Immediate occupancy	Damage limitation	Low damage	T_R=72
Slight	Operational	-	Integrity	T_R=50

Table 2.2 - Damage levels, performance criteria and return periods.

2.4 Structural health monitoring of historic structures in seismic regions

Historical structures and infrastructures are inevitably subjected to ageing effects and require expensive maintenance acts and surveillance against accidental events in order to preserve them. The availability of a permanent assessment of the structural conditions is essential to assure an appropriate level of reliability and safety for the historical constructions. In addition to the traditional methods, in the last three decades new experimental procedures have been developed in order to provide widespread and accurate information about the structural performance and integrity.

Farrar and Worden [27] define structural health monitoring (SHM) as a process which involves the periodic monitoring of a structure through measurements, the extraction of features symptomatic to the phenomena under investigation and their statistical analysis to determine the actual state of the system. A diagnostic monitoring system is therefore the result of the integration of several sensors, devices and auxiliary tools, like:

- a measurement system;
- an acquisition system;
- a data processing system;
- a communication/warning system;
- an identification/modelling system;
- a decision making system.

Even if it is based on innovative measuring, analysing, modelling and communication techniques, SHM shares the same goals of traditional methods. In fact, the diagnostic monitoring can be considered as an extension of the well-established investigation practices since it integrates these novel technologies in a unique smart system. SHM tries to overcome the limitations of traditional visual inspections.

There are several reasons which let prefer an automatic monitoring system working in real or at least nearly-real time rather the investigations performed periodically. First of all, it is a matter of economic convenience. Visual inspections must be carried out by high qualified personnel with a periodic recurrence which is not related to the actual state of the structure. Sometimes expensive service equipment and the complete closure or, at least, the partial limitation of the structure usability is required to perform the inspections. A permanent monitoring system is much more cost-effective on a long period of time because of the amortization of the initial costs due to the ideation, design and execution. This issue is even more stressed in the case of the historical constructions because, differently from ordinary structures, they do not have a limited life-cycle to accomplish their function.

The traditional survey methods are affected by a large series of technical drawbacks. Visual inspections are generally performed with a periodicity too spaced in time which risks affecting their predictive nature. Moreover, they are neither exhaustive, because they do not allow to identify hidden defects or the invisible effects of an on-going damage process, nor objective, because the estimation is related to the subjective judgement of an expert who can be fallible. More specific and accurate non-destructive testing (NDT) techniques [28] are carried out off-line and usually only after the damage has been located. This means that in the meanwhile an excessive level of deterioration could have been reached. Nevertheless, non-destructive estimations are performed in a local manner and so can provide useful information referred to a limited portion of the structure.

Modern diagnostic monitoring systems were born with the prerogative to overcome these limitations providing an exhaustive depiction of the structural health state and easing the plan of maintenance and restoring interventions. Recently, the vibration-based damage assessment has proved its potentialities in different applications. Modal properties have been successfully used for the damage identification in real existing structures. Many issues require further investigation and still represent challenges that have to be undertaken. A new philosophy must be pursued, which is aware of the importance of a reasoned design of the monitoring system. It must integrate a sensors network which is capable of operating a continuous surveillance and providing reliable analyses based on different information sources. The environmental and operating conditions variability must be taken into account too.

2.4.1 SHM technologies

The SHM is the result of the integration of disciplines and technologies flatly different each other, but reciprocally essential. The elements which combine to bring about the conception and the execution of a reliable monitoring system can be subdivided in: experimental, analytical and information technologies [29].

2.4.1.1 Experimental technologies

Belong to this category all the techniques and the methodologies developed throughout the years to sense the structural response. Further classifications employ different discrimination criteria, for instance the monitoring goal, the static or dynamic acquisition of the measurements, the destructive or not-destructive nature of the tests, the monitoring duration, etc. Table 2.3 provides a brief summary of the most important experimental technologies classified according to their static, dynamic or hybrid nature.

Static test	Non-destructive static tests	Measurement of the response in a limited portion of the structure
	Destructive static tests	Laboratory or in-situ destructive tests: expensive and difficult to generalise
Dynamic test	Non-destructive dynamic tests	Vibration analysis carried out to extract modal parameters, using different excitation sources (ambient, hammer, drop of weight, electro-dynamic actuators).
	Permanent monitoring	Measuring system permanently installed on a structure, acquiring periodically different quantities.
Hybrid test	Geometric monitoring	Laser scanning, global positioning systems, photogrammetry, remote sensing technologies in order to track geometry changes.
	Non-destructive evaluation	Non-destructive technologies able to detect hidden construction details, defects or damage or to determine the physical and chemicals properties of the materials.

Table 2.3 - Principal experimental technologies used in SHM.

2.4.1.2 Analytical technologies

Analytical technologies provide the tools to simulate the structural behaviour in the operational condition and in the presence of damage. The analytical investigation is essential both in the diagnostic and in the prognostic phase. In the former, the causes of damage are identified from the correlation between the observed and the analysed symptoms; in the latter their evolution is tracked. The structural modelling can be geometric or numerical, but always deterministic. Recently, a stochastic approach is spreading, in particular in the case of historical constructions, where the uncertainties referred to the mechanical properties of the materials, the boundary and connection conditions and the strong non-linear effects cannot be disregarded. However, further research is still required and in most of the cases a deterministic approach, whose uncertainties are handled varying the parameters within some bounds and according to the results of sensitivity analyses, is preferred.

Anyway, the adopted model must be able to capture the geometric, mechanical and boundary properties in an extensive manner and to simulate the damage scenarios of the structure in a reliable way. In this sense the experimental data represent a precious source of information to assess the model and its predictive capabilities and to calibrate it accordingly. The instruments which contribute to the analysis of the structural behaviour are:

- elaboration and representation tools (CAD, databases);
- instrumental survey technologies (photogrammetry, laser scanner ...);
- numerical modelling methods.

Computer-aided design (CAD) packages represent the first step in the analytical modelling and structural identification. A geometric conceptualisation of the construction is achieved through a three-dimensional depiction. Photographic and photogrammetric pictures can be used to define the dimensions accurately and to capture all the most significant details. The numerical modelling can be performed according to different levels of resolution and consequent computational efforts. The most commonly implemented methods are the finite element method (FEM), the boundary element method (BEM) and the finite difference method (FDM).

The FEM is generally the adopted solution because of its large spread and the availability of many commercial packages. Three-dimensional models are essential to deal with the geometric complexity of the historical constructions and to capture their peculiar loading and damage conditions. Some of the most common objectives for modelling are:

- to provide a basis for the monitoring system design;
- to provide a term of comparison for the results of the structural identification;
- to serve as a baseline for assessing any future change in the structural conditions and to define some reliable warning thresholds;
- to evaluate the vulnerability of the structure and to identify those critical elements which require a focused attention in the monitoring;
- to evaluate possible causes of observed damage, dysfunctional behaviours or performance deficiencies;
- to forecast the evolution of observed damage scenarios according to the actual structural conditions;
- to design restoration interventions on the structure.

The available methods of analysis are subdivided between linear and nonlinear. Most software packages offer a wide range of linear analyses, which include static analyses under various combinations of stationary loads, eigenvalue analysis to determine the modal properties of the structure and dynamic analyses. Non-linear analyses are characterised by the capability to take the mechanical and the geometric non-linearity effects into account. Non-linear analyses allow to estimate the position of possible cracks openings and the consequences of other damage phenomena in the structure and to track their evolution in time. The drawbacks of this class of analyses are the high computational effort and calculation time, the difficulties in the results interpretation and the huge sensitivity of the accuracy of the analysis to the parameters selection. The choice of the wrong set of analysis and mechanical parameters can affects drastically the results and lead to a complete misunderstanding of the investigated phenomena.

2.4.1.3 Information technologies

The collection of the experimental data is useless if it is not efficiently supported by data recording, storage, analysis, interpretation and representation techniques. The complex process of the raw data transformation into information useful for the monitoring purposes must be accomplished in a systematic manner, adopting appropriate methodologies. Automatic procedures for the collection, classification and storage of experimental data have to be defined in the structural monitoring design phase. The number and typology of the employed sensors, the number of the available acquisition channels and the selected sampling rate are conditioning elements for the implementation of these techniques.

Some of the most common operations performed after data acquisition are here summarised:

- Data formatting: signals referred to similar measurements must be formatted in the same way in order to ease the automation of the data processing.
- Data classification: the acquisition results must be organised according to a hierarchic framework and an appropriate nomenclature in order to ease the data storage and collection.
- Preliminary check: to verify the correctness of the signals acquisition and data formatting and classification.
- Data communication: from the acquisition system to a remote unit through cabled lines or wireless systems. The rapid development of more and more powerful data communication technologies will allow the spread of real-time monitoring system and will reduce their costs.
- Data storage: must follow pre-established criteria which take into account the subsequent need for signal processing, visualisation of the results and information sharing. The adoption of databases management programs and systems responsible for data storage can be helpful.
- Data pre-processing: it includes all those operations aimed at the preparation of data for the subsequent feature extraction. They are generally referred as "data cleansing" and some examples are the filtering aimed at the noise reduction, the decimation, the mean removal or the outliers removal. The data cleansing process can also involve the selection of data to pass or reject according to the knowledge gained during human inspections of the test equipment. Sometimes also the conversion of a time-history to a spectrum by means of the Fourier transformation is considered as pre-processing operation to reduce the data dimension.
- Feature extraction: consists in the extraction of useful information for the monitoring purposes from the pre-processed data. The usefulness of the extracted features is referred to the damage sensitivity. For instance, in the case of the dynamic monitoring, the acquired acceleration time-histories can be processed by means of modal identification techniques in order to extract the modal properties of the structure. These features are sensitive to the structural changes produced by damage and can be employed for its assessment.
- Information interpretation: is the formulation of the diagnosis about the structural health based on the information inferred from the experimental data. Several statistical tools can be used to accomplish this task. The more common are multivariate correlation analyses, Bayesian methods, pattern recognition methods, neural networks and genetic algorithms. The comparison with the results obtained from the analytical simulation is worthy too.
- Results presentation: is the synthesis of the information obtained from the monitoring and it depends on the nature of the results and the pursued goals. The results are generally portrayed by means of plots which show the evolution of the extracted features or through indices assumed to be sensitive to the structural

health. The simplicity and the immediacy of the representation are essential requirements.
- Decision making: is the conversion of the results into a decision whether any action need to be taken and, eventually, what kind of action should be.

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