

Chapter 1

SEISMIC DESIGN PRINCIPLES IN STRUCTURAL CODES

1.1 INTRODUCTION

Earthquake Engineering is the branch of engineering aiming at mitigating risks induced by earthquakes with two objectives: i) to predict the consequences of strong earthquakes on urban areas and civil infrastructures; ii) to design, build and maintain structures that are able to withstand earthquakes in compliance with building codes.

Researchers and experts working within emergency management organizations (e.g. the civil protection) actively work on the first issue. On the contrary, structural designers focus their attention and efforts on the second objective. With this regard, it should be noted that the seismic design philosophy substantially differs from the design approaches conventionally adopted for other types of actions, raising difficulties to structural engineers less confident with seismic engineering. Indeed, broadly speaking, for quasi-static loads (e.g. dead and live loads, wind, snow, etc.) the structure should behave mostly elastically without any damage until the maximum loads are reached, while in case of seismic design it is generally accepted that structures can experience damage because they should perform in the plastic range for seismic events. The philosophy of structural seismic design establishes the performance levels that properly engineered structures should satisfy for different seismic intensities, which can be summarized as follows:

- prevent near collapse or serious damage in rare major ground shaking events, which are called in the following Ultimate Limit State seismic action or ULS seismic action;
- prevent structural damage and minimize non-structural damage in occasional moderate ground shaking events;
- prevent damage of non-structural components (such as building partitions, envelopes, facilities) in frequent minor ground shaking events.

Hence, the most meaningful performance indexes for seismic resistant structures are the amount of acceptable damage and the repair costs. Owing to the unforeseeable nature of seismic actions, it is clear that damage control is very difficult to be quantified by code provisions, especially because it is related to acceptable levels of risk. The challenge for efficient design of seismic resistant structures is to achieve a good balance between the seismic demand (namely the effect that earthquakes impose on structures) and the structural capacity (namely the ability to resist seismic induced effects without failure). However, the quantification of different types of damage (structural and non-structural) associated to the reference earthquake intensity (e.g. frequent/minor, occasional/moderate, and rare/major) and the definition of relevant operational design criteria are still open issues that need clarification and further studies.

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This chapter describes and discusses the concept of capacity design in the light of existing seismic codes, illustrating the evolution of seismic design principles throughout time, and explains the criteria that form the basis of EN 1998-1:2004 (CEN, 2004a), henceforth denoted as EC8-1.

1.2 FUNDAMENTALS OF SEISMIC DESIGN

1.2.1 Capacity design

It is generally acknowledged that structural safety depends on the ductility that the structural system can provide against the design loads. Indeed, ductility represents the capacity of a mechanical system (e.g. a beam, a structure, etc.) to deform in the plastic domain without substantially reducing its bearing capacity.

In seismic design of structures it is generally not economical or possible to ensure that all the elements of the structure behave in a ductile manner.

Inevitably, a dissipative (ductile) structure comprises both dissipative (ductile) elements and non-dissipative (brittle) ones. In order to achieve a dissipative (ductile) design for the whole structure, the failure of the brittle elements must be prevented. This may be done by prioritizing structural elements strength, which will lead to the prior yielding of ductile structural elements, preventing the failure of brittle structural elements. This principle is known as "capacity design". Capacity design may be explained by considering the chain model, introduced by Paulay and Priestley (1992) and depicted in Figure 1.1a, whereby the chain represents a structural system made of both ductile elements (e.g. the ring "1") and brittle zones (e.g. the ring "i").

According to non-seismic design procedures for quasi-static loads (hereinafter referred to as "direct design"), the design force is the same for all elements belonging to the chain, because the applied force is equal for all rings, being a system in series. Hence, the design resistance $F_{y,i}$ is the same for all elements. Under this assumption, the yield resistance of the ductile chain $F_{y,1}$ is equal or even slightly larger than $F_{y,i}$.

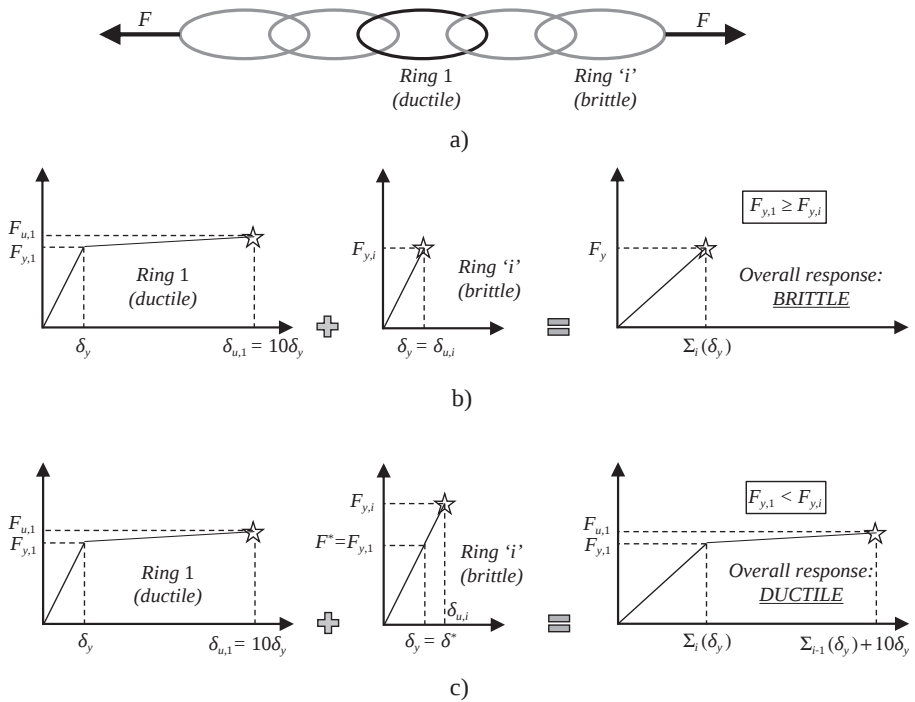


Figure 1.1 – Ductility of a chain with brittle and ductile rings

As shown in Figure 1.1b, with the direct design approach the system cannot develop strength larger than F_y and the ultimate elongation of the chain is given as

$$\delta_u = \sum_i \delta_y = 5\delta_y \quad (1.1)$$

According to capacity design principles, in order to improve the ductility of the chain, some rings should be designed with ductile behaviour and lower strength, as is the case of ring “1” in Figure 1.1c. The remaining rings “ i ” that are brittle should be designed to provide a resistance $F_{y,i}$ larger than the maximum resistance $F_{u,1}$ exhibited by the ring “1” beyond yielding. The ductile ring “1” behaves as a sacrificial element, i.e. a ductile fuse, which filters the external actions and limits the transfer of forces into the brittle elements. Hence, the maximum force that the chain can sustain is equal to the maximum resistance $F_{u,1}$ of the ductile ring “1”. It is interesting to observe that the beneficial improvement of the capacity design methodology is the increase of displacement capacity, given as follows:

$$\delta_u = \sum_{i=1} \delta_y + 10\delta_y = 4\delta_y + 10\delta_y = 14\delta_y \quad (1.2)$$

4 Comparing equations (1.1) and (1.2), it can be easily recognized that the collapse displacement of the chain is significantly larger than that obtained by adopting the direct design approach.

This trivial example allows to understand that the brittle elements represent protected zones that must be designed to resist larger forces than those supported by the ductile elements. Those larger forces do not directly depend on the external applied loads but they are obtained from the maximum capacity of the connected ductile elements. However, it should be emphasized that the external forces are used to design the dissipative elements, which establish the threshold of structural strength.

Concerning the practical application to building structures, this methodology leads the structural designers to work on two different schemes for the same structure, as follows:

- 1) elastic behaviour with the calculation of the relevant internal forces F_{Ed} to design the dissipative elements. Hence, following an elastic analysis, the ductile structural elements should satisfy the following check:
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$$F_{ductile,Rd} \geq F_{Ed} \quad (1.3)$$

In addition to strength, the ductile elements must possess a ductility corresponding to the chosen ductility class. The ductility is provided by using appropriate structural details and different materials and specific design principles for specific types of structures;

- 2) inelastic response with design of non-dissipative (i.e. brittle) elements on the basis of the plastic strength of the connected dissipative parts. Hence, in order to prevent their failure, brittle elements must be sized so that they present an over strength with respect to the capacity of the ductile elements, as follows:

$$F_{brittle,Rd} \geq \Omega F_{ductile,Rd} \quad (1.4)$$

where Ω is a coefficient (> 1.0) that takes into account different aspects that may lead to ductile elements strengths larger than the design ones (strain hardening phenomena, material strength larger than the nominal values, etc.).

This twofold approach is the basic characteristic of capacity design and represents the main distinctive difference with respect to direct design for quasi-static actions. The example shown in Figure 1.1 also allows understanding that the common belief of non-seismic designers, which consider that the excess of strength is always beneficial and safe, may dramatically impair the non-linear response of a structure either by overdesigning the fuse elements or, with more serious consequences, by inaccurate quality control of the material properties that results in larger strength for the dissipative elements (e.g. a steel element conceived as fuse with grade S355 is supplied with higher grade as S460). The consequence of such events is clear, namely the failure of the system because the hierarchy of resistance is not complied with.

In case of steel structures the best way to dissipate energy is to exploit the tensile capacity of the material, which can be obtained by enforcing plasticity into specific zones called plastic hinges that can involve either flexural, tensile or shear mechanisms depending on the type of adopted structural scheme (e.g. moment resisting frame, concentrically or eccentrically braced frame), while preserving the rest of the structure from damage.

1.2.2 Seismic design concepts

Two substantially different concepts can be used to design structures located in seismic areas, which correspond to two different structural behaviours:

- Concept (a): low-dissipative (and/or non-dissipative) behaviour;
- Concept (b): dissipative behaviour.

The difference between dissipative and non-dissipative behaviours is dictated by both the ductility and energy dissipation capacity that the structure can provide. The ductility represents the capacity to deform in the plastic domain without substantially reducing its bearing capacity. However, there are other properties that significantly influence the seismic performance, namely the displacement and dissipative capacity. These properties are not synonyms, but all of them contribute towards a satisfactory seismic behaviour. Some examples may be helpful to clarify the differences between ductility, displacement and dissipative capacity.

Figure 1.2 shows the load-deflection response curves of two different frames subjected to monotonically increasing horizontal loads. The maximum strength F_y of the frame corresponds to the yield strength and/or stability limit load, and the deformation capacity δ_u corresponds to the sudden decrease of the strength that can be caused by the rupture of steel material, global and/or local buckling of steel members and/or crushing of concrete. Even though the strength of both frames is identical, the one with the response curve shown in Figure 1.2a represents a ductile behaviour, which is substantially different from that of Figure 1.2b that corresponds to a brittle performance. Indeed, the first structure is characterized by a larger ductility $\mu = \delta_u/\delta_y$ and also a larger displacement capacity δ_u , which is the capacity of the structural system to experience large ultimate displacements. Also, the amount of energy absorbed by the frame shown in Figure 1.2a before it reaches the limit deformation δ_u is larger than that of the frame shown in Figure 1.2b. In light of the remarks in section 1.2.1, the response of the frame shown in Figure 1.2a is more efficient for an earthquake resisting structure.

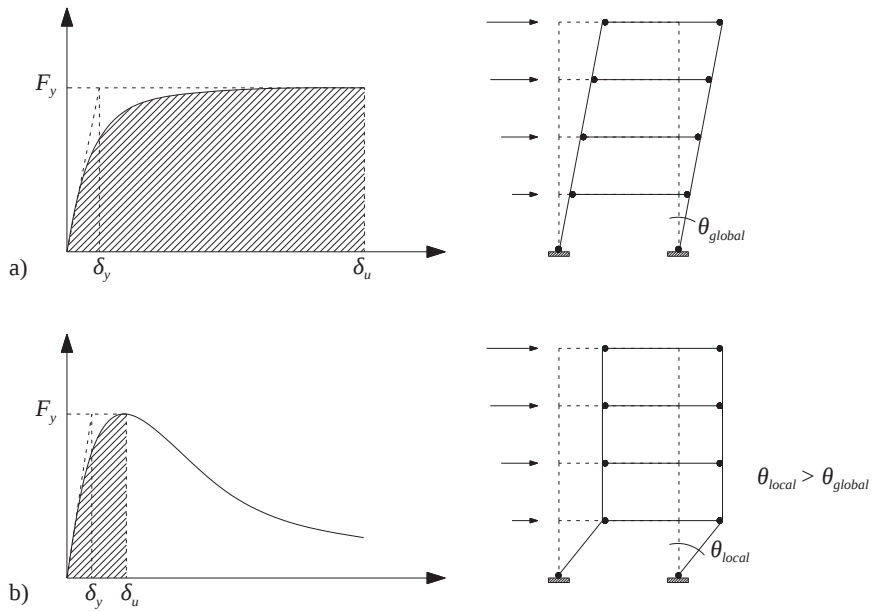


Figure 1.2 – Ductility of frames: a) high and b) poor displacement capacity

However, adequate seismic behaviour also depends on the shape of the cyclic response of both the structure and the dissipative zones. With this regard, Figure 1.3 shows two examples of hysteresis loops of frames under repeated horizontal load, having the same monotonic response and displacement capacity δ_u . In these cases, in addition to the effects indicated above, the shape of the hysteresis loops also depends on the number of loading cycles, since deformation phenomena associated with fatigue caused by the repeated loading may have some effect on it. The frame shown in Figure 1.3a dissipates larger energy before failure than the one in Figure 1.3b, thus providing a better seismic performance, the energy being the area within a loop. Hence, dissipative capacity can be defined as the ability to dissipate energy by means of stable and compact hysteretic loops.

Ductile and dissipative structures are very convenient because they avoid brittle phenomena and lead to less expensive constructions. In order to exploit the ductility, ductile structures are generally designed to resist seismic forces substantially smaller than those needed to obtain an elastic response

under seismic action corresponding to the Ultimate Limit State (ULS). However, plastic deformations imposed by the seismic action must not exceed the deformation capacity of the structure in the plastic domain, in order to prevent excessive damage that may compromise the stability against gravity loads and/or make unfeasible a subsequent refurbishment. Thus, the minimum strength F_y of the structure against lateral forces that is needed to avoid excessive damage is directly related to the structure's deformation capacity in the plastic domain. For the ULS seismic action, different strength/ductility combinations can be determined that satisfy the design demands.

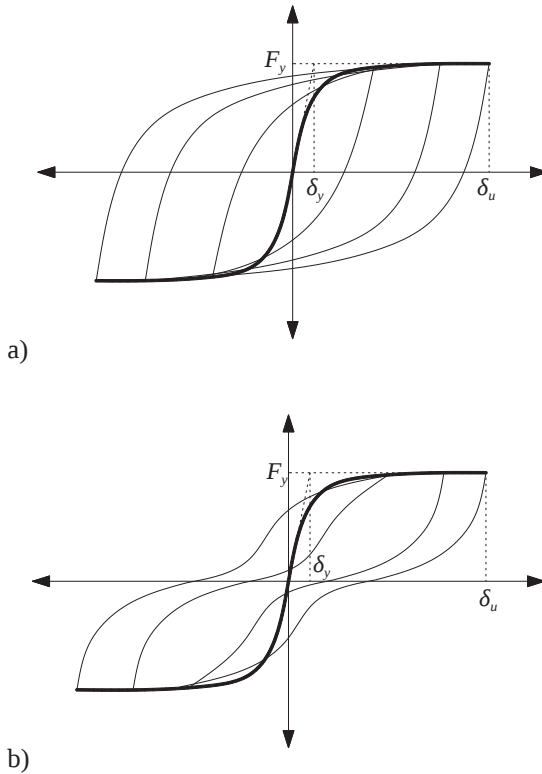


Figure 1.3 – Dissipative capacity of frames: a) high and b) poor energy absorption

The fundamental relationship between the strength of the structure to lateral forces (F_y) and the displacement demand (δ_{Ed}) imposed to the structure by a given level of the seismic action is presented in Figure 1.4a.

For the same the same displacement capacity, the lower is the strength of the structure to lateral forces (F_y), the higher is the ductility demand ($\mu_{Ed,i} = \delta_{Ed}/\delta_{y,i}$) imposed to the structure. Thus, the more ductile and dissipative structures may be designed to lower lateral forces that can be determined by scaling the elastic forces by the so-called behaviour factor q , which strictly depends on the structural system (see Figure 1.4b).

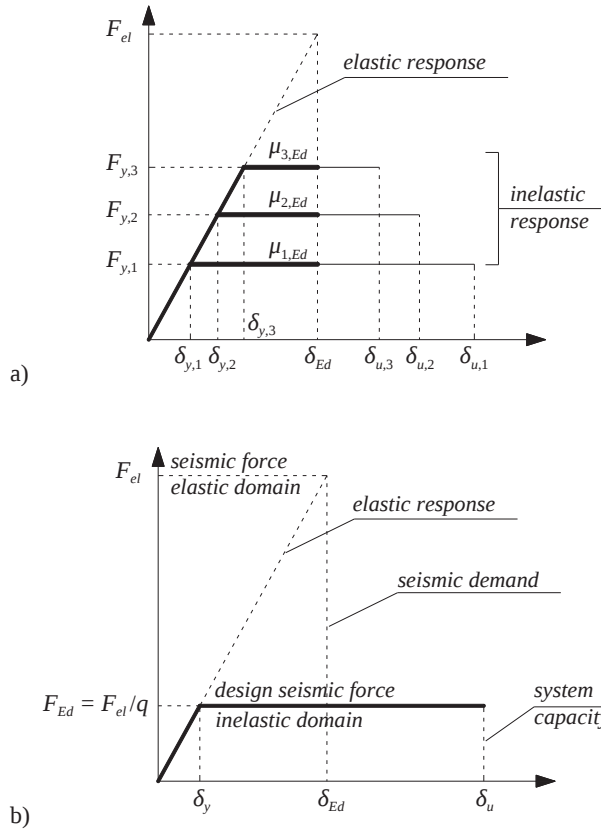


Figure 1.4 – Strength vs. displacement demand relationship

Modern codes like EC8-1 give the possibility to choose different ductility levels for a structure, providing different ductility classes. It is understandable that choosing a ductility class instead of another has direct consequences on the design process. In case of EC8-1 there are at least two major features. The first is the value of the design seismic load, which is obtained by scaling the elastic design forces by a behaviour factor q (see Figure 1.4b). The structures

that are designed to behave in a more ductile way (i.e. on a higher ductility class) have higher values of the behaviour factor q , and, consequently, lower design seismic forces. The second consequence of choosing a ductility class is the necessity of providing a certain ductility level to the structure. To achieve this purpose, the codes provide specific detailing and design requirements for all structural materials (e.g. steel, reinforced concrete, timber, etc.) and relevant types of structures (e.g. moment resisting and braced frames, structural walls, etc.) compliant with each ductility class.

This approach cannot be adopted for structural typologies that do not provide any ductility and/or dissipative capacity, such as the so-called low-dissipative (brittle) structures. Indeed, because the force exhibits a sudden decrease beyond their elastic limit, these structures must be designed to remain almost elastic under the ULS seismic action. This corresponds to using a behaviour factor q close to unity. Because these structures do not exploit any plastic behaviour, their design may be carried out according to the direct design procedures used for non-seismic conditions. Therefore, seismic design provisions (for example EC8-1) are used only to determine the seismic loading, and the ULS structural checks are carried out according to general structural design standards (for example, the EN 1993 series in case of steel structures).

Designing a structure as dissipative or low-dissipative is a decision of the structural engineer. Fundamentally, any structure can be designed according to one of the two concepts. Generally, choosing the design concept accounts for economical aspects, depending on the type of the structure and the seismic area. With this regard, it should be noted that structural details and design demands necessary to provide ductility and dissipative behaviour may lead to higher constructional and design effort. Therefore, if the elastic (non-reduced) seismic forces acting on the structure are relatively small and the design is mainly governed by non-seismic load conditions, the low-dissipative design principle of the structure can be economically used. By omitting the design demands meant to provide a ductile global behaviour, the design process will be simplified and will lead to reduced material consumption.

However, for many types of structures, the seismic action represents a very severe design action, more critical than the other loading conditions, and providing an elastic response of the structure under the effect of the design seismic action at ULS will lead to excessive size of the structural elements and, consequently, to an excessive material consumption. Hence, in those cases,

the dissipative design concept of the structure should be adopted, exploiting the structural ductility by both designing the structure for reduced seismic forces smaller than those corresponding to an elastic response and detailing the dissipative zones in order to provide the required larger ductility.

Consequently, it can be stated that the low-dissipative design concept is reasonably suitable and economic for small seismic forces, while a dissipative design one is more economic and effective for large seismic forces.

An interesting consideration can be drawn considering the nature of seismic forces. Indeed, actions induced by earthquakes are inertial forces, generated by the acceleration upon the structure masses as a result of the seismic motion imposed to the base of the structure. Therefore, the seismic forces will have smaller values for light structures. On the contrary, the seismic forces will have important values for structures with larger mass. An example of a light structure, for which the low-dissipative design principle is suitable, are single storey steel warehouses. These constructions are characterized on one hand by a relative small self weight and on the other hand by small live loads. In contrast, typical examples of structures for which the dissipative design principle is suitable are multi-storey buildings, because of the large masses resulting from permanent (reinforced concrete slabs) and live loads.

1.3 CODIFICATION OF SEISMIC DESIGN

1.3.1 Evolution of seismic design codes

Seismic engineering is a relatively new branch of structural engineering, since the first criteria were developed only at the beginning of the 20th century, while the most important modern concepts were established in the last 50 years (Gioncu and Mazzolani, 2002). In Europe, the first seismic design concepts were introduced by Gustave Eiffel at the beginning of last century, who modelled the earthquake action through an equivalent wind load. San Francisco, in California, was rebuilt after the 1906 great seismic event with this model by assuming a 1.4 kPa equivalent wind pressure to estimate seismic actions.

In Europe, the first quantitative seismic code was developed by an Italian Government Commission following the 1908 Messina–Reggio earthquake,

which killed 160000 people. A report was issued giving a procedure that, for the first time, proposed to estimate the forces induced by the earthquake on a structure as a percentage of its weight. Accordingly, the first floor earthquake equivalent force was estimated to be $1/12$ of the weight above, changing to $1/8$ of the weight for the upper floors. This method promoted an equivalent static approach, which is still in use nowadays in most design codes.

In Japan, after the 1923 earthquake in Kanto, which killed 140000 people, the Home Office of Japan adopted a design regulation that stipulated the use of horizontal equivalent static forces equal to 10 % of the building weight, limiting also the height of the buildings.

In the USA, the concept of lateral seismic forces proportional to mass was introduced after the Santa Barbara earthquake in 1925 and the 1933 Long Beach earthquake and the buildings then had to be designed to carry lateral forces equal to 7.5 % and 10 % of their dead load for rigid and soft soils, respectively. The influence of structural flexibility and the number of floors on the design forces was recognized by the Los Angeles city code in 1943. The San Francisco recommendations gave the first provisions to take into account the influence of the dynamic properties of a structure by relating the seismic forces to the fundamental period of vibration. These simple concepts were based on grossly simplified physical models, engineering judgment and a number of empirical coefficients. For many years, the standard design methodology was based on models where the structures were considered as elastic systems and the earthquake actions as static loads.

The modern concepts of response spectrum and plastic deformation were introduced by Benioff (1934) and Biot (1941). The concept of ductility and energy dissipation capacity was proposed for the first time by Tanabashi in 1935, according to whom the earthquake resistance capacity of a structure should be measured by the amount of energy that the structure can dissipate before collapse. The first attempts to combine these two aspects, namely the response spectrum and the dissipation of seismic energy through plastic deformations, were made by Housner (1956, 1959), who used the velocity response spectra of the elastic system to have a quantitative evaluation of the total amount of energy input that contributes to the building response, by assuming the hypothesis that the energy input, responsible for the damage in the elastic-plastic system, is identical to that in the elastic system (Akiyama, 1985). The first studies on inelastic spectra were carried out by Veletos and Newmark (1960) who obtained

the maximum response deformation for an elastic-perfectly plastic structure. At that stage, the response spectrum became a standard measure of the demand of the ground motion. Despite being based on a simple Single-Degree-Of-Freedom (SDOF) linear system, the concept of the response spectrum was extended to Multi-Degree-Of-Freedom systems (MDOF), non-linear elastic systems and inelastic hysteretic systems. Indeed, the response spectrum represents a powerful design tool because it gives a simple and direct indication of the overall displacement and acceleration demands of the earthquake ground motion, for structures having different period and damping characteristics, without the need to perform detailed numerical analysis. Newmark and Hall proposed a new concept in 1969 (Newmark and Hall, 1969, 1982), by constructing spectra based on accelerations, velocities, and displacements, in short, medium and long period ranges, respectively. More recently, different design methodologies have been elaborated for near-field and far-field regions. Indeed, the ground motions in near-field regions are qualitatively different from those of the commonly used far-field earthquake ground motions. For near-field earthquakes the importance of the higher vibration modes cannot be neglected; therefore, the use of the equivalence of multi-degree-of-freedom systems with only one degree-of-freedom gives inaccurate results. In the early 1970s, an important change in seismic design took place thanks to advent of personal computers and the availability of a large number of softwares able to perform static and dynamic analyses in the elastic and plastic ranges. This new technology allows to obtain more refined results, giving researchers the possibility to improve the design spectra methodology by means of a more correct calibration of design values. At the same time, the seismic behaviour of structures may be evaluated in a more precise way thanks to time-history methodologies, by using real recorded accelerograms.

Consequently, by the end of 1970s, the second generation of seismic design codes was developed, which started to take into account both the dynamic amplification and the energy dissipation properties in the estimation of the statically equivalent seismic design forces. However, the design and calculation procedures remained quite rough and did not allow for particularities between the behaviour of structures made of different materials and of different lateral force resisting systems (Bisch, 2009).

The significant economic losses and human casualties that resulted from the Northridge and Kobe earthquakes (that occurred in 1994 and 1995, respectively), even if the no-collapse objective had been met for many

structures (Bommer and Pinho, 2006), led to the development of a new generation of prescriptive seismic design codes (i.e. the third one) with the aim to improve the criteria for overstrength and ductility and to qualify structural details in dissipative zones. Moreover, significant progress was done in using advanced methodologies of structural analysis, such as non-linear static and, particularly, dynamic analyses.

This category of codes established minimum requirements for safety through the specification of prescriptive criteria that regulate acceptable materials of construction. Moreover, they identified structural and non-structural systems approved for seismic applications, specifying the required minimum levels of strength and stiffness and controlling the relevant details. Although these prescriptive criteria were intended to result in buildings capable of providing certain levels of performance, the actual performance of individual building designs is not assessed as part of the traditional code design process. As a result, the performance capability of some buildings designed to these prescriptive criteria could be better than anticipated by the code, while the performance of others could be worse.

However, the added value of these codes was the introduction of a novel design philosophy that is known as “Performance-Based Seismic Design (PBSD)”, which synthesises the significant concept that multi-level design criteria have to be applied to achieve a set of design objectives. The main peculiarity consists in correlating the structural performance at various limit states to the probability of occurrence of the earthquake action that reaches the intensity required to induce the corresponding failure modes (Mazzolani and Gioncu, 2000). The combination of the structure performance level with the specific level of ground motion represents the performance design objective. The aim of this new approach is to provide criteria for selecting the appropriate structural system and for detailing both structural and non-structural components, so that for specified levels of earthquake intensity the structural damage will be constrained within pre-defined limits in order to achieve a good balance between adequate safety levels and economy (Mazzolani and Piluso, 1996).

PBSD has been developed within the activities of SEAOC Vision 2000 Committee (SEAOC, 1995). The acceptability of varying levels of damage was determined on the basis of the consequences of this damage to the user community and the frequency of occurrence of such damage (see Figure 1.5). The four following performance levels were proposed:

- Fully operational, in which no damage occurs and the consequences to the building and its user community are negligible;
- Operational, in which moderate damage to non-structural elements and contents, and light damage to structural elements may occur; however, the damage does not compromise the safety of the building for occupancy;
- Life safe (damage state), in which moderate damage to both structural and non-structural elements occurs; nevertheless, both lateral stiffness and strength of the structure to resist additional lateral loads is reduced, but some margin against collapse remains;
- Near collapse (extreme state), in which the lateral and vertical load resistance of the building are substantially compromised; aftershocks could result in partial or total collapse of the structure.

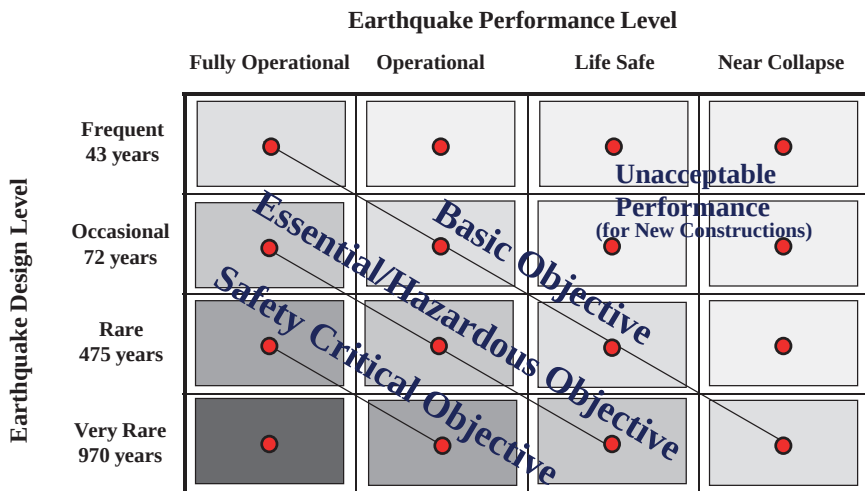


Figure 1.5 – Seismic performance design objective matrix
(SEAOC, 1995)

In addition, SEAOC Vision 2000 specified four earthquake design levels: frequent, occasional, rare and very rare, which are characterised by return periods equal to 43, 72, 475 and 970 years, respectively. It is clear that it is accepted that structures may fail at more severe seismic intensities (Bertero 1996).

Three design levels are defined in Figure 1.5. The “Basic Objective” applies to the majority of buildings. For more critical structures, higher

performance objectives would be the reference, like the “Essential/Hazardous Objective” or even higher, the “Safety Critical Objective”.

In contrast to prescriptive design approaches, PBSD ideally provides a systematic methodology for assessing the performance capability of a building, system or component. Indeed, PBSD explicitly evaluates how a structure will likely perform considering the potential hazard it is likely to experience, accounting for both uncertainties related to the quantification of the potential hazard and random and epistemic uncertainties that are related to the assessment of the actual building response. The use of PBSD allows to design both new buildings or to upgrade existing buildings with an accurate and realistic understanding of the risks and economic loss that may occur in case of future earthquakes. In addition, PBSD provides a framework to determine the levels of safety and property protection with the corresponding costs, thus allowing to evaluate the thresholds acceptable by building owners, tenants, lenders, insurers, regulators and other decision makers based upon the specific needs of a project.

However, this framework was far from being entirely implemented in seismic codes, owing to its complexity and the lack of guidelines. An initial suggestion by Bertero (1996) was to carry out a preliminary design of structures taking into account only two performance levels, such as the operational and the life safety, then check the structure for all the intermediate levels in order to assess the design acceptability. In this way a compromise was reached between the traditional design philosophy and this new philosophy, which was convenient considering the widespread use of the traditional design practice.

About ten years later, the fourth generation of seismic codes was developed. Indeed, FEMA 445 (2006) opened the way to the full implementation of Performance Based Seismic Design (PBSD) methods in current design practice, which represents a significant improvement with respect to the previous seismic codes.

FEMA 445 started a work plan devoted to cover the shortcomings and criticisms of the previous third generation codes, with a threefold action, as follows:

- To revise the discrete performance levels (as those defined in SEAOC Vision 2000), to develop new performance measures (e.g. repair costs, casualties, and time of occupancy interruption) that more efficiently relate to the decision-making needs of stakeholders,

and that communicate these losses in a way that is more meaningful to stakeholders;

- To develop accurate guidelines to carry out both analytical and numerical procedures for the prediction of actual building response; and to create procedures for estimating probable repair costs, casualties, and time of occupancy interruption, for both new and existing buildings;
- To develop a framework for performance assessment that properly accounts for, and adequately communicates to stakeholders, the limitations in the ability to accurately predict response and uncertainty in the level of earthquake hazard.

This activity culminated in two new codes, i.e. FEMA 695 (Quantification of Building Seismic Performance Factors) and FEMA 750 (Recommended Seismic Provisions for New Buildings and Other Structures), that are at the present time the most advanced seismic regulations.

In particular, FEMA 695 provides a standard procedural methodology that allows quantifying the inelastic response characteristics and performance of typical structures and verifying the adequacy of the structural system provisions to meet the design performance objectives. Such a methodology directly accounts for the potential variations in structural configurations of structures designed, and for the variation in ground motion to which these structures may be subjected. In addition, the behavioural characteristics of structural elements are validated on the basis of available experimental data.

Within this continuously progressing codification process, Eurocode 8 bridges in-between the third and fourth generation of codes. At the present time, also in Europe there is a six years action plan devoted to update the current version of Eurocode 8. With this regard, CEN/TC250/SC8, which is the official body devoted to the maintenance and development of all parts of Eurocode 8, established specific working groups (WG), one per chapter of the code, aiming to overcome the criticisms to the code and to implement the recent outcomes from scientific research. One crucial specific aspect relates to the quantification of the seismic hazard and the improvement and harmonization of the European Seismic Zonation (Solomos *et al*, 2008). In light of Figure 1.5, it should be emphasized that while many parts of central and northern Europe present low seismicity with low peak ground accelerations for a rare seismic

event (475 years return period or, equivalent, 10 % probability of exceedance over 50 years), see Figure 1.6, crucial infrastructures such as nuclear power plants should be designed for 10000 years return periods. In this case, a major part of Europe may be affected. Seismic design provisions for critical structures (e.g. dams, nuclear power plants, etc.) are out of the scope of both EC8-1 and this handbook. However, the seismic zonation provided by EC8 may be considered as the basis for the definition of the seismic input.

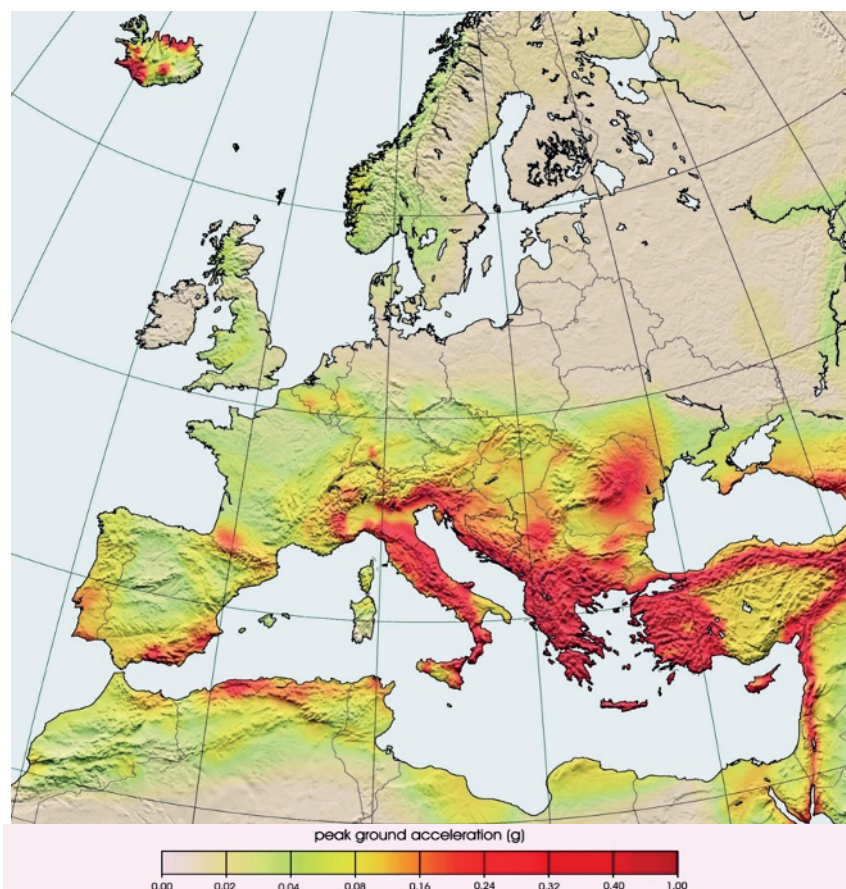


Figure 1.6 – ESC-SESAME European-Mediterranean seismic hazard map for the peak ground acceleration with 10 % probability of exceedance in 50 years for rock site (Solomos *et al*, 2008)

It should be noted that the seismic zonation prescribed by EC8 defines zones for which the reference peak ground acceleration hazard on a “rock”

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severe or even moderate earthquake activity, buildings have suffered extensive damage and even total collapse. As a consequence, the building design codes increase constantly the seismic demands, to cover the uncertainty of hazard quantification by improving the structural response capacity through accuracy of design and enhanced technical solutions. On the other hand, the design methodology includes enhanced models of analysis and calculation tools, associated with more relevant performance criteria, in order to obtain a better prediction and control of structural response. There are three practical and efficient strategies to reduce the seismic vulnerability, i.e.:

- (1) reducing seismic design forces;
- (2) enhancing structural damping;
- (3) adjusting the structural response to seismic demand (see Figure 1.8).

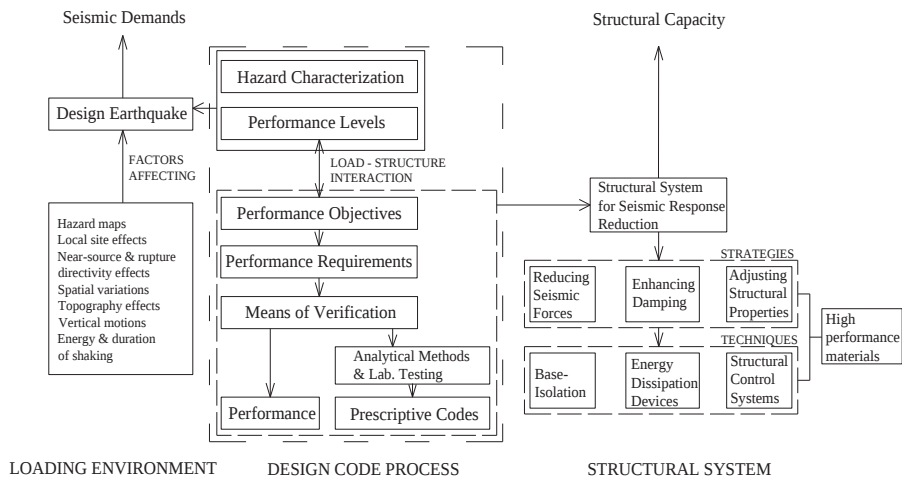


Figure 1.8 – Seismic protection strategies (Rai, 2000)

The first approach, which implies the method of structural isolation, is very efficient, but expensive for existing buildings. The principle behind isolation is to change the natural period of the structure, substantially decoupling a structure from the ground motion input and therefore reducing the resulting inertia forces that the structure must resist. This is done by the insertion of devices allowing a relative displacement between the structure base and the ground or between the upper part of the structure (the roof for instance) and the bottom part of the structure. These devices can be for

instance energy absorbing material such as rubber bearings. They will reduce the amount of seismic forces transmitted to the structure.

The second strategy applies passive energy dissipation devices and has shown great potential for seismic hazard mitigation for civil engineering structures. They can significantly enhance the structural performance by reducing inelastic deformation demands on the primary lateral load resisting system and the drift, acceleration and velocity demands on non-structural components. Passive devices can be categorized as rate independent devices (e.g., hysteretic or friction systems) and rate dependent devices (e.g., viscous or viscoelastic systems), where the force output of the latter type of devices is dependent on the rate of applied deformation. In the USA, the NEHRP recommendations allow the structural engineer to utilize passive damping devices to attain performance similar to that of conventional lateral load resisting systems. The design methods used for structures with passive energy dissipation systems are usually based on an approximate or iterative approaches. Experiments and load tests are often required to evaluate and validate these design methods.

The third strategy is in contrast to the approaches mentioned earlier. Indeed, both base isolation and passive devices do not allow changing and adjusting the structural response evolutively with the seismic signal. On the contrary, there is an innovative expanding class of systems referred to as ‘smart’ or active control systems that allow modifying the vibrational response with the variation of seismic excitation. Different smart techniques have been proposed in recent years that involve adjusting lateral strength, stiffness and damping of the structure during the earthquake to reduce the structural response. Many studies and some field applications have emphasized their potential in reducing the structural response. However, many serious problems are still far from being solved such as the time delay in the control actions, modelling errors, inadequacy of sensors and controllers, structural nonlinearities and reliability, and finally the high operational costs. Therefore, this design strategy is not robust enough and still requires research to be validated and to be implemented in seismic codes.

EC8-1 allows the design of buildings with base isolation (in its chapter 10), but this part of Eurocode 8 is still more informative than applicative, and requires additional guidelines. In Europe, there is also a code specifically devoted to the design, characterization and the acceptance criteria of anti-

seismic devices, namely EN 15129 (CEN, 2009). This standard specifies the functional requirements and general design rules for the seismic situation, material characteristics, manufacturing and testing requirements, as well as evaluation of conformity, installation and maintenance requirements, but does not give guidelines for structural design and/or criteria for retrofitting intervention with seismic devices on existing buildings. Therefore, in Europe there are surely needs for further codification and technical support documents for design, in order to support their use in practice.

In light of recent scientific findings and the current status of North American codes previously described, it is clear that EC8-1 needs to be updated and improved. With this regard, the Joint Research Centre (JRC) in 2007 elaborated a EN document (Pinto *et al*, 2007), which summarises the current needs to achieve improved Design Guidelines for seismic protection in Europe. The following objectives of further engineering research aiming to improve European seismic regulations were identified:

- Development of a common methodology to evaluate the earthquake hazard in Europe;
- Development of an assessment and strengthening methodology for more economical and safe solutions for the seismic retrofit of the existing building stock in European earthquake prone areas;
- Development of strengthening techniques of low intrusive effect for application in monuments, historical buildings and other structures;
- Seismic design and upgrading of mechanical, electric and other types of equipment used in the lifelines and industry.

More recently, with respect to steel structures, the Technical Committee 13 - Seismic Design, of the European Convention for Constructional Steelwork (ECCS), issued the document P131/2013 entitled “Assessment of EC8 provisions for seismic design of steel structures” that identifies and proposes the needs and subjects that should be addressed by the drafting team of the ongoing revision of the eurocodes and chapter 6 – Steel structures of EC8-1 in particular. As a non-exhaustive list, the following items were identified for development and inclusion in the future version of EC8-1, chapter 6:

- New criteria for choice of material, in terms of overstrength and toughness;
- Better definition of local ductility: relevant criteria, consideration of class 4 sections;
- Connections in dissipative zones: prequalification criteria;
- New structural systems: new typologies, definition of q factors for them; dissipative components working as fuse devices (such as Buckling Restrained Braces, removable links in Eccentric Braced Frames); systems with re-centering capacity;
- Seismic design of structures in low seismicity zones.

