Design of Steel Structures for Buildings in Seismic Areas

Eurocode 8: Design of Structures
for Earthquake Resistance
Part 1: General Rules, Seismic Action
and Rules for Buildings

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FOREWORD

There are many seismic areas in Europe. As times goes by, regional seismicity is better known and the number of places where earthquake is an action to consider in design increases. Of course, there are substantial differences in earthquake intensity between regions and the concern is much greater in many areas of Italy, for instance, than in most places in Northern Europe. However, even in Northern Europe, for structures for which a greater level of safety is required, like Seveso industrial plants, hospitals and public safety facilities, seismic design can be the most requiring design condition.

Designing for earthquake has original features in comparison with design for classical loading like gravity, wind or snow. The reference event for Ultimate Limit State seismic design is rare enough for an allowance to permanent deformations and structural damages, as long as people's life is not endangered. This means that plastic deformations are allowed at ULS, so that the design target becomes a global plastic mechanism. To be safe, the latter requires many precautions, on global proportions of structures and on local detailing. The seismic design concepts are completely original in comparison to static design. Of course, designing for a totally elastic behaviour even under the strongest earthquake remains possible but, outside of low seismicity areas, this option is generally left aside because of its cost.

This book is developed with a constant reference to Eurocode 8 or EN 1998-1:2004; it follows the organization of that code and provides detailed explanations in support of its rather dry expression. Of course, there are many other seismic design codes, but it must be stressed that there is nowadays a strong common thinking on the principles and the application rules in seismic design so that this book is also a support for the understanding of other continents codes.

Chapter 1 explains the principles of seismic design and their evolution throughout time, in particular the meaning, goals and conditions set forward by capacity design of structures and their components, a fundamental aspect of seismic design.

Chapter 2 explains the general aspects of seismic design: seismic actions, design parameters related to the shape of buildings, models for the analysis, safety verifications. Methods of analysis are explained in an exhaustive way: theoretical background, justifications of limits and factors introduced by the code, interest and drawbacks of each method, together with occasionally some tricks to facilitate model making and combination of load cases.

Chapter 3 focuses on design provisions specific to steel structures: ductility classes, requirements on steel material, structural typologies and design conditions related to each of them; an original insight on design for reparability is also included.

Chapter 4 provides an overview about the best practice to implement the requirements and design rules for ductile details, particularly for connections in moment resisting frames (MRF), concentrically braced frames (CBF) and eccentrically braced frames (EBF), and for other structural components like diaphragms.

Chapter 5 describes the guidance provided for design assisted by testing by EN 1990 and the specific rules for tests, a necessary tool for evaluating the performance characteristics of structural typologies and components in the plastic field and in cyclic/dynamic conditions.

Chapter 6 illustrates and discusses the design steps and verifications required by EN 1998-1 for a multi-storey Moment Resisting Frame.

Chapter 7 and 8 do the same respectively for buildings with CBF's and EBF's.

Chapter 9 presents three very different examples of real buildings erected in high seismicity regions: one tall building, one industrial hall and one design using base isolation. These examples are complete in the sense that they show the total design, where seismic aspects are only one part of the problem. These examples are concrete, because they illustrate practical difficulties of the real world with materials, execution, positioning…

The concepts, design procedures and detailing in seismic design may seem complex. This publication explains the background behind the rules, which clarify their objectives. Details on the design of the different building typologies are given, with reference to international practice and to recent research results. Finally, design examples and real case studies set out the design process in a logical manner, giving practical and helpful advice.

This book will serve the structural engineering community in expanding the understanding and application of seismic design rules, and, in that way, constitute a precious tool for our societies safety.

André Plumier
Honorary Professor, University of Liege

Preface

This manual aims to provide its readers with the background and the explanation of the main aspects dealing with the seismic design of steel structures in Europe. Therefore, the book focuses on EN 1998-1 (usually named part 1 of Eurocode 8 or EC8-1) that is the Eurocode providing design rules and requirements for seismic design of building structures. After 10 years from its final issue, both the recent scientific findings and the design experience carried out in Europe highlight some criticisms. In the light of such considerations, this book complements the explanation of the EC8-1 provisions with the recent research findings, the requirements of renowned and updated international seismic codes (e.g. North American codes and design guidelines) as well as the design experience of the Authors. Although the manual is oriented to EC8-1, the book aims to clarify the scientific outcomes, the engineering and technological aspects rather than sticking to an aseptic explanation of each clause of the EC8-1. Indeed, as shown in Chapter 4, the proper detailing of steel structures is crucial to guarantee adequate ductility of seismic resistant structures and the current codes does not give exhaustive guidelines to design ductile details since it only provides the fundamental principles. In addition, the practice of earthquake engineering significantly varies between European regions, reflecting the different layouts of each national seismic code as well as the level of knowledge and confidence with steel structures of each country. With this regard, a large number of European engineers believe that steel structures can withstand severe earthquakes without requiring special details and specifications as conversely compulsory for other structural materials like reinforced concrete and masonry. This belief direct results from the mechanical features of the structural steel, which is a high performance material, being stronger than concrete but lighter (if comparing the weight of structural members) and also very ductile and capable of dissipating large amounts of energy through yielding when subjected to cyclic inelastic deformations. However, although the material behaviour is important, the ductility of steel alone is not enough to guarantee ductile structural response. Indeed, as demonstrated by severe past earthquakes (e.g. Northridge 1994, Kobe 1995 and Christchurch 2011) there are several aspects ensuring good seismic behaviour of steel structures, which are related to (i) the conceptual design of the structure, (ii) the overall sizing of the member, (iii) the local detailing and (iv) proper technological requirements as well as ensuring that the structures are actually constructed as designed.

Therefore, this book primarily attempts to clarify all these issues (from Chapter 1 to 4) for European practising engineers, working in consultancy firms and construction companies. In addition, the examples of real buildings (see Chapter 9) are an added value, highlighting practical and real difficulties related to both design and execution.

This design manual is also meant as a recommended textbook for several existing courses given by the Structural Sections of Civil Engineering and Architectural Engineering Departments. In particular, this manual is oriented to advanced students (i.e. those attending MSc programmes) thanks to the presence of various calculation examples (see Chapter 6, 7 and 8) that simplify and speed up the understanding and the learning of seismic design of EC8 compliant steel structures. In addition, research students (i.e. those attending PhD programmes) can find useful insights for their experimental research activities by reading Chapter 5, which provides some guidance and discussion on how performing experimental tests of structural typologies and components in cyclic pseudo-static and dynamic conditions.

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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.

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 $F_{y,i}$.

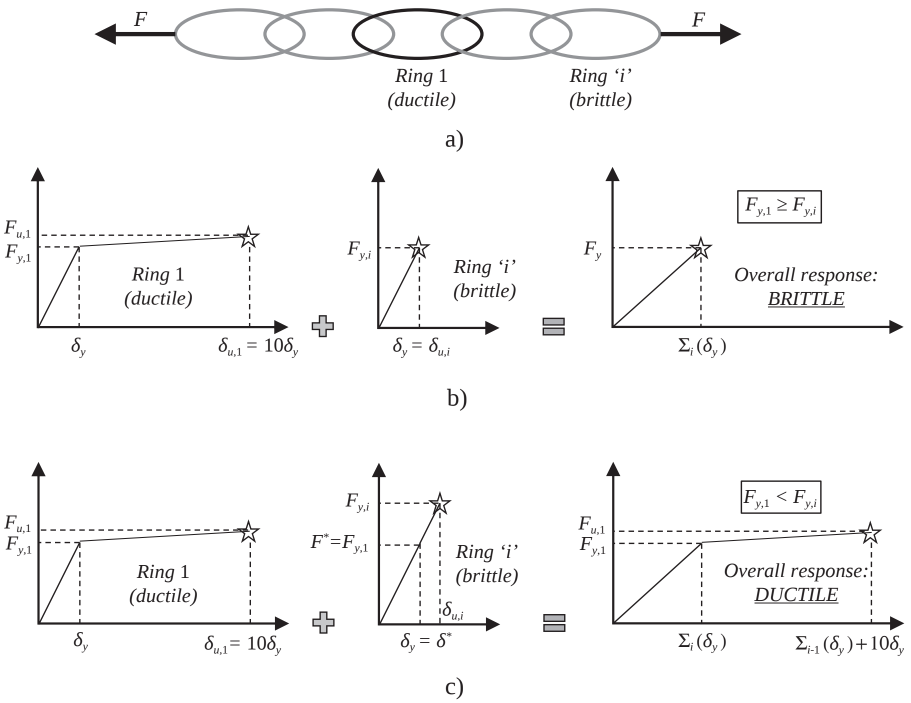


Figure 1.1 – Ductility of a chain with brittle and ductile rings

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

$$\delta_u = \sum_i \delta_y = 5\delta_y \tag{1.1}$$

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:

$$\delta_u = \sum_{i-1} \delta_y + 10\delta_y = 4\delta_y + 10\delta_y = 14\delta_y \tag{1.2}$$

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:

$$F_{ductile,Rd} \geq F_{Ed} \tag{1.3}$$

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:

$$F_{brittle,Rd} \geq \Omega F_{ductile,Rd} \tag{1.4}$$

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 l.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 l.2b. In light of the remarks in section 1.2.1, the response of the frame shown in Figure l.2a is more efficient for an earthquake resisting structure.

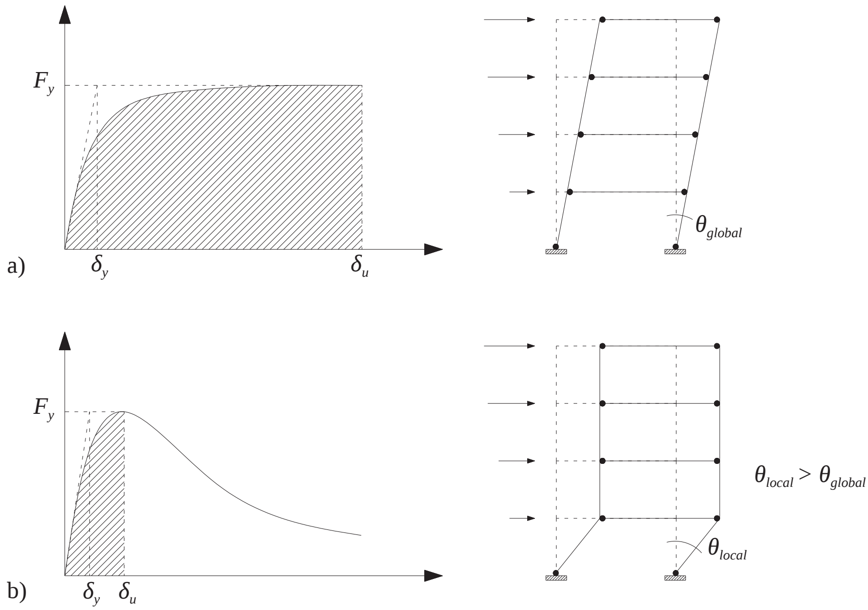
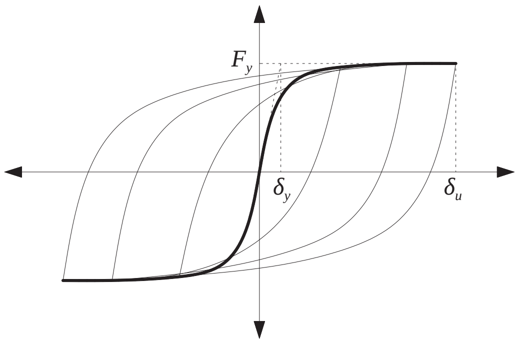


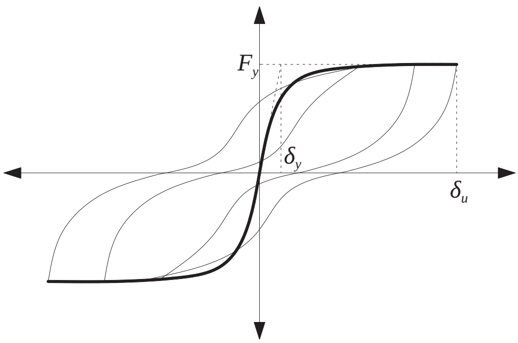
Figure 1.2 – Ductility of frames: a) high and b) poor displacement capacity

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.



a)



b)

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).

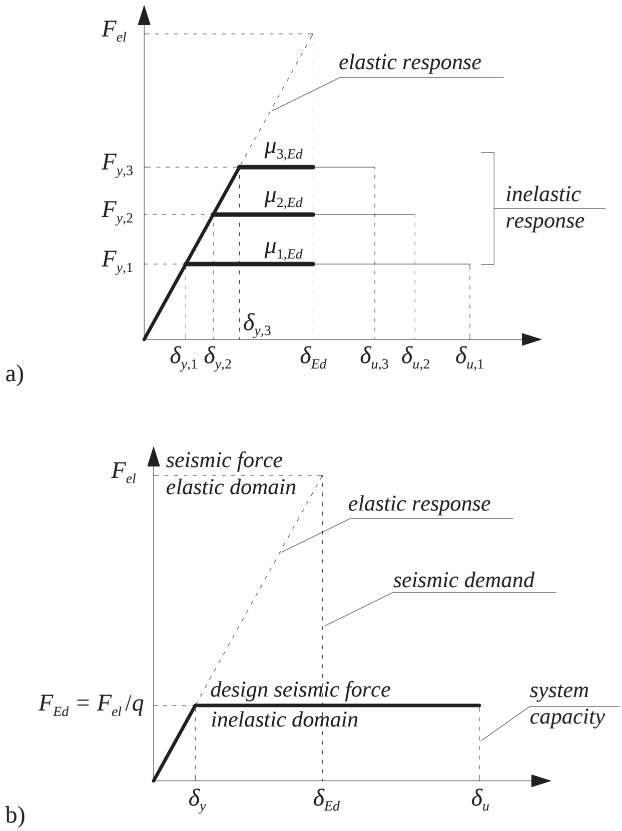


Figure 1.4 – Strength vs. displacement demand relationship

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,