Chapter 1
General Principles for the Design of Concrete Buildings for Earthquake Resistance

Chapter 1 presents the requirements posed by modern seismic codes and standards for the protection of life and property in new building designs and highlights the means provided for their fulfilment. The requirements and design rules provided in the European Standard for the seismic design of new buildings – EN 1998-1:2004, termed also Part 1 of Eurocode 8 – are given certain emphasis and compared to their US counterparts. These Eurocode 8 rules are elaborated further in Chapter 5 in the context of the process for the detailed design of new concrete buildings for earthquake resistance.

Chapter 1 gives also an overview of a new thinking towards more comprehensive coverage of the seismic performance needs of owners and occupants over the lifetime of the building. This thinking is currently penetrating newly emerging codes and standards for the seismic evaluation and upgrading of existing substandard buildings, including EN 1998-3:2005 (also known as Part 3 of Eurocode 8). The requirements and rules provided in this latter European Standard for the seismic assessment and retrofitting of existing buildings are further elaborated in Chapter 6.

1.1 Seismic Performance Requirements for Concrete Buildings

1.1.1 The Current Situation: Emphasis on Life Safety

Traditionally, introduction and enforcement of structural design codes and standards has been the responsibility of competent Authorities, with public safety as the overriding consideration. Accordingly, traditional seismic design codes or standards for buildings aim at protecting human life by preventing local or global collapse under a single level of earthquake. The no-(local-)collapse requirement normally refers to a rare seismic action, termed "design seismic action". In most present codes the "design seismic action" for ordinary structures is conventionally chosen as the one having a 10% probability to be exceeded in a conventional working life of 50 years, or 0.2% in a single year. This corresponds to a mean return period of 475 years for the "design seismic action".

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Within a single-tier design framework, enhanced safety of facilities that are essential or have large occupancy is normally achieved by modifying the hazard level (the mean return period) of the "design seismic action". The seismic action is multiplied times an "importance factor", γ_I. By definition, $\gamma_I = 1.0$ for structures of ordinary importance (buildings of "Importance Class" II in Eurocode 8). For buildings whose collapse may have unusually large social or economic consequences (large occupancy buildings, such as schools or public assembly halls, etc.) or for facilities housing institutions of cultural importance (e.g., museums), Eurocode 8 recommends a value $\gamma_I = 1.2$ (buildings of "Importance Class" III in Eurocode 8). It recommends $\gamma_I = 1.4$ for buildings which are essential for civil protection during the immediate post-earthquake period: hospitals, fire or police stations, power plants, etc. (categorised as "Importance Class" IV). For buildings of minor importance for public safety (i.e., belonging in "Importance Class" I, comprising agricultural and similar buildings) Eurocode 8 recommends a value $\gamma_I = 0.8$.

1.1.2 Performance-Based Requirements

Already in the 1960s the international earthquake engineering community was fully aware of the property loss that may be caused by frequent seismic events and their other economic consequences. Recognising that it is not feasible to avoid any damage under very strong earthquakes, the Structural Engineers Association of California (SEAOC) adopted in its 1968 recommendations the following requirements for seismic design:

"Structures should, in general, be able to:

- Resist a minor level of earthquake ground motion without damage.
- Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.
- Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage."

Major earthquakes that hit developed countries in the second half of the 1980s and the first half of the 1990s caused relatively few casualties but very large damage to property and economic losses. In response to this, "Performance-based earthquake engineering" emerged in the SEAOC Vision 2000 document and developed into the single most important idea of recent years for seismic design or retrofitting of buildings (SEAOC 1995).

"Performance-based engineering" focuses on the ends, notably on the ability of the engineered facility to fulfil its intended purpose, taking into account the consequences of its failure to meet it. Conventional structural design codes, by contrast, are process-oriented, emphasising the means, namely the prescriptive, easy to apply, but often opaque rules that disguise the pursuit of satisfactory performance. These rules have been developed over time as a convenient means to provide safe-sided, yet economical solutions for common combinations of building layout, dimensions and materials. They leave limited room for the designer to exercise judgement and creativity and do not provide a rational basis for innovative designs that benefit from recent advances in technology and structural materials.

"Performance-based earthquake engineering" in particular tries to maximise the utility from the use of a facility by minimising its expected total cost, including the short-term cost of the work and the expected value of the loss in future earthquakes (in terms of casualties, cost of repair or replacement, loss of use, etc.). One would like to take into account all possible future seismic events with their annual probability and carry out a convolution with the corresponding consequences during the design working life of the facility. However, this is not practical. Therefore, at present "performance-based earthquake engineering" advocates just replacing the traditional single-tier design against collapse and its prescriptive rules, with a transparent multi-tier seismic design, meeting more than one discrete "performance levels", each one under a different seismic event, identified through its annual probability of exceedance and termed "seismic hazard level". Pairing off all "performance levels" considered for a particular case with the associated "seismic hazard levels" is termed, in performance-based earthquake engineering, "performance objective".

Each "performance level" is normally identified with a physical condition of the facility, well-described together with its possible consequences: likely casualties, injuries and property loss, continued functionality, cost and feasibility of repair, expected length of disruption of use, cost of relocation of occupants, etc. Commonly four "performance levels" are identified:

(i) "Operational"
(ii) "Immediate occupancy"
(iii) "Life-safety" and
(iv) "Near collapse".

The definition of these "performance levels" is roughly as follows:

"Operational": The facility has suffered practically no structural or non-structural damage and can continue serving the original intention of its design with little disruption of use for repairs. Continuous operation is supported either by undamaged lifelines or by back-up systems. Any repair that is necessary can take place in future without disruption of occupancy or use.

"Immediate occupancy": The facility can return to full use, as soon as utility systems are back in operation and cleanup is complete. The structure itself is very lightly damaged: some yielding of reinforcement may have taken place and concrete cracking may be visible, but there are no residual drifts or other permanent structural deformations. The risk to life is negligible. The structure retains fully its pre-earthquake strength and stiffness. Its ability to withstand future earthquakes, including aftershocks, is not diminished. Non-structural components and systems may have minor damage (e.g. distributed cracking in infill walls) that can be easily and economically repaired at a later stage.

"Life-safety": The structure, or any parts of it, do not collapse, retaining integrity and residual load capacity after the earthquake. The structure is significantly damaged and may have moderate permanent drifts, but retains its full vertical load-bearing capacity and sufficient residual lateral strength and stiffness to protect life even during strong aftershocks. Non-structural components are damaged, but do not block evacuation routes or cause life-threatening injuries by falling. Sometimes reparability is economically questionable and demolition may be preferable.

"Near collapse": The structure is heavily damaged, at the verge of collapse of several gravity load-carrying elements in a storey, or even of total collapse. It may have large permanent drifts and retains little residual strength and stiffness against lateral loads, but its vertical elements can still carry the (quasi-)permanent gravity loads. Most non-structural elements (e.g. infill walls) have collapsed. There is substantial, but not full, life safety, as falling hazards may cause life-threatening injury. The building is unsafe for use, as it may collapse in a strong aftershock. Repair may not be technically feasible and certainly is not economically sensible.

Sometimes, reference is made to two more performance levels: "Damage onset", as a performance level before "Operational" associated with absolutely no structural or non-structural damage; and "Reparable", as a performance level between "Immediate occupancy" and "Life-safety", associated with structural or non-structural damage that is not only technically, but also economically, reparable.

Different performance criteria are also defined for the verification of structural or non-structural elements under the various performance levels. Criteria for structural or non-structural damage are normally expressed in terms of deformation limits. For example, performance level (i) ("Operational") may be identified with "yielding" of structural members, while performance level (iv) ("Near collapse") is often associated with near exhaustion of member "ultimate" deformation, signalling loss of lateral load capacity. Damage limitation criteria for non-structural cladding or partitions that follow the deformations of the structural frame are normally expressed in terms of interstorey drift limits. For equipment mounted or supported on the structure, limits relevant to damage may be expressed in terms of response accelerations at the support points of the equipment.

The discrete hazard levels normally paired off with the four main performance levels listed under (i)–(iv) above for the design of ordinary (i.e., standard occupancy) new buildings, are:

1. a "frequent" earthquake, expected to take place during the conventional working life of the building, having therefore a mean return period much shorter than 50 years (e.g., around 25 years);
2. an "occasional" earthquake, not expected during the conventional working life of the building, with a mean return period between 75 and 200 years;
3. a "rare" earthquake, with a mean return period of about 500 years; and
4. a "very rare" or "maximum considered" earthquake, with quoted values of the mean return period in the order of 1000–2500 years.

According to this idea, the "performance objective" for structures of ordinary importance is to meet performance level (i) under hazard level (1), (ii) under (2), etc.

If higher performance is desired, or for critical facilities, an “enhanced objective” may be selected – e.g. performance level (ii), or even (iii), under hazard level (1), etc.

Note that, depending on the slope of the seismic hazard curve, at any given site certain aspects of the design may be governed by the fulfilment of one performance level under the corresponding hazard level. The other performance levels will be met then automatically at the associated hazard levels. If this applies in general to all types of buildings at a given geographic location or region, then a four-tier performance-based seismic design may degenerate there into a fewer-tier (e.g., a two-tier) one.

Performance-based seismic design serves better the interests and objectives of owners, by allowing more rational decision-making, with explicit verification of performance levels related to property loss and operation of the facility under frequent or occasional earthquakes. It may also provide more flexibility in conceptual design, as collapse prevention under very rare events is explicitly verified, instead of indirectly designed against by explicit verification only at the “life safety” level and using capacity design as a safeguard against collapse under much stronger earthquakes (see Section 1.3). On the other hand, a full-fledged performance-based design process may be arduous and complex. Besides, there is a liability issue to be resolved: the designer is protected to a certain extent against liability claims or other charges for property loss, casualties, etc., in an unforeseeable future event, if he or she has strictly adhered to all rules of a current-generation prescriptive code, which is opaque about the intended performance objective. This may not be the case anymore in a performance-based design context, with explicit and transparent performance objectives which the owner or the courts may interpret as guaranteed. For all these reasons, there is still a long way to go before seismic design codes for new buildings adopt a full-fledged performance-based approach. Such an approach has been adopted, though, in guidelines and standards for the seismic assessment and retrofitting of existing buildings, as it is there that the inherent flexibility of the approach can best bear fruits to accommodate the specific interests, objectives and means of owners. Moreover, buildings not designed to modern-day seismic codes normally do not possess structural features serving as safeguards against collapse under very strong earthquakes (e.g., a layout and a hierarchy of strengths that prevent concentration of deformation demands in a small part of the structural system). Therefore, older buildings require explicit verification against such an outcome.

1.1.3 Performance-Based Seismic Design, Assessment or Retrofitting According to Eurocode 8

In Europe performance levels in seismic design, assessment or retrofitting are associated to, or identified with, Limit States of the structure. The Limit State concept appeared in Europe in the 1960s, to define states of unfitness of the structure for its intended purpose (CEB 1970, Rowe 1970). Limit States concerning the safety of people or of the structure are termed Ultimate Limit States. Those concerning the normal function and use of the structure, the comfort of its occupants, or damage to property (mainly to finishes and non-structural elements) are called Serviceability Limit States. Intermediate Limit States may also be considered (CEB 1988b). According to the Eurocode "Basis of Structural Design" (CEN 2002) the Limit States approach is the backbone of structural design for any type of action, including the seismic one.

Part 1 of Eurocode 8 (CEN 2004a) provides for a two-tier seismic design of new buildings, with the following explicit performance levels ("Limit States"):

1. No-(local-)collapse, which is considered as the Ultimate Limit State against which the structure should be designed according to the Eurocode "Basis of Structural Design" (CEN 2002). It entails protection of life under a rare seismic action, through prevention of collapse of any structural member and retention of structural integrity and residual load capacity after the event.
2. Damage limitation, which plays the role of the Serviceability Limit State against which the structure should be designed according to CEN (2002). The aim is mitigation of property loss in frequent earthquakes, through limitation of structural and non-structural damage. After such an earthquake structural elements are supposed to have no permanent deformation, retain their full strength and stiffness and need no repair. Non-structural elements may suffer some damage, which can be easily and economically repaired at a later time.

The no-(local-)collapse performance level is achieved by dimensioning and detailing structural elements for a combination of strength and ductility that provides a safety factor (in the order of 1.5–2) against substantial loss of lateral load resistance.

The damage limitation performance level is achieved by limiting the overall deformations (lateral displacements) of the building to levels acceptable for the integrity of all its parts (including non-structural ones). More specifically, interstorey drift ratios (defined as the difference between the mean lateral displacements of adjacent storeys divided by the interstorey height) are limited to the following values:

(i) 0.5%, if the storey has brittle non-structural elements attached to the structure (notably, ordinary masonry infills);
(ii) 0.75%, if the storey's non-structural elements are ductile; or
(iii) 1%; when there are no non-structural elements that follow the deformations of the structural system.

The two explicit performance levels – (local-)collapse prevention and damage limitation – are pursued under two different seismic actions. The seismic action under which (local) collapse should be prevented is the "design seismic action". The one for which damage limitation is pursued is called the "damage limitation seismic action". Within the Eurocode philosophy of national competence on issues of safety and economy, the hazard levels for these two seismic actions are left to national determination. For structures of ordinary importance, Part 1 of Eurocode 8 recommends:

1. a “design seismic action” having 10% probability of being exceeded in 50 years (a mean return period of 475 years); and
2. a “damage limitation seismic action” with 10% exceedance probability in 10 years (mean return period: 95 years).

Although not explicit, an additional performance objective in buildings designed to provide earthquake resistance by dissipating energy is to prevent global collapse during a very strong and rare earthquake (performance level (iv) in Section 1.1.2 under hazard level (4)). This implicit performance objective is pursued via systematic and across-the-board application of capacity design, which imposes a hierarchy of strengths that permits full control of the inelastic response mechanism (see Section 1.3).

Following the example of the US standard for seismic rehabilitation (ASCE 2007) and its draft predecessors, Part 3 of Eurocode 8 for assessment and retrofitting of buildings (CEN 2005a) has fully adopted the “performance-based” approach. It provides for three different performance levels (termed Limit States):

1. “Damage Limitation” (DL), corresponding to “Immediate Occupancy”: The structure has no permanent drifts; its elements have no permanent deformations, retain fully their strength and stiffness and do not need repair. Members are verified to remain elastic.
2. “Significant Damage” (SD), corresponding to “Life safety” and to the (local-)collapse prevention performance level to which new buildings are designed according to Part 1 of Eurocode 8. The structure is significantly damaged, may have moderate permanent drifts, but retains some residual lateral strength and stiffness and its full vertical load-bearing capacity. Repair may be uneconomic. The verifications should provide a margin against member ultimate capacities.
3. “Near Collapse” (NC), similar to “Collapse prevention” in the US: The structure is heavily damaged, may have large permanent drifts, retains little residual lateral strength or stiffness, but vertical elements can still carry the gravity loads. In the verifications, a member may approach its ultimate force or deformation capacity.

The “Seismic Hazard” levels for which the three Limit States should be met are chosen either nationally through the National Annex to this part of Eurocode 8, or by the owner if the country leaves the choice open. The Eurocode itself gives no recommendation, but mentions that the performance objective recommended as suitable for ordinary new buildings is a 225 year earthquake (20% probability of exceedance in 50 years), a 475 year event (10% probability in 50 years), or a 2475 year one (2% probability of being exceeded in 50 years), for the DL, the SD or the NC "Limit State", respectively. Countries (or the owners, if the country lets the choice to them) have the authority to decide whether all three Limit States will be verified, or whether checking one or two of them at the corresponding seismic hazard level suffices.

1.1.4 Performance-Based Design Aspects of Current US Codes

In the NEHRP provisions (BSSC 2003) seismic design of new buildings is for a single level of ground motion, namely for two-thirds of the Maximum Considered Earthquake (MCE). This is the "design seismic action" in the US. The MCE is given by the USGS Seismic Hazard Maps from the USGS/BSSC 97 project (Frankel et al. 1996, 1997). These maps are also used by almost all recent nationally applicable US documents. They map the values of the 5%-damped elastic response spectral acceleration in the acceleration-controlled region, S_{as} (which is equal to 2.5 times the effective peak acceleration, *EPA*) and at a period of 1 s (S_{a1}, from which the velocity-controlled spectral region is derived). National and regional maps (at a scale of 1:500.000–1:5.000.000) are given for the MCE, which is defined for this purpose as 1.5 times the characteristic event produced by well known active faults every few hundred years. Where no major active faults can be identified, the values of S_{as} and S_{a1} with 2% probability of being exceeded in 50 years (i.e., with mean return period of 2500 years) is used. Factors are given for the conversion of the values of S_{as} and S_{a1} over firm rock to other types of ground.

For structures of ordinary importance the Life Safety performance level is required under the design seismic action of two-thirds of MCE. If this performance objective is fulfilled, it is deemed that collapse prevention is indirectly achieved under the 1.5-times stronger MCE and that immediate occupancy is expected under a frequent event with 50% probability of being exceeded in 50 years (mean return period of 72 years). Facilities which are essential for post-earthquake recovery or contain hazardous substances are designed for 1.5-times higher forces (through a 1.5-times smaller force reduction factor), implying Life Safety performance under the MCE. Such structures are claimed to indirectly achieve the Immediate Occupancy performance level under frequent earthquakes. Structures with increased public hazard, owing to large occupancy or limited ability of occupants to evacuate (medical or daycare facilities, schools, jails), are designed for 25% higher forces than ordinary ones and believed to fulfill intermediate performance objectives.

The performance objectives achieved by other than ordinary structures through the SEAOC '99 recommendations are less clear: they provide just for 25% increased design forces for essential or hazardous facilities.

Note that the importance of the structure is taken into account only in the performance under the single level of design action considered and does not affect the design seismic action. This is also evident from the fact that the importance factor does not enter in the calculation of storey drifts – calculated and checked under the design seismic action for life protection and not under a more frequent event for damage limitation.

1.2 Force-Based Seismic Design

1.2.1 Force-Based Design for Energy-Dissipation and Ductility

For the no-(local-)collapse requirement to be met for the "design seismic action" the structure does not need to remain elastic under this action. That would have required a lateral force resistance close to 50% of the building's weight. Although technically feasible, this is economically prohibitive. It is also completely unnecessary, as the earthquake is a dynamic action and imparts to the structure a certain total energy input and certain displacement and deformation demands, but not a demand to sustain specific forces. So, current codes for earthquake-resistant design allow structures to develop significant inelastic deformations under the design seismic action, provided that the integrity of individual members and of the structure as a whole is not impaired. The design approach for this is still based on forces, but its real aim is to impart to the structure capacity for energy dissipation and ductility.

Force-based seismic design is against physical reality. It is the deformation that causes a structural member to lose its lateral load resistance. It is lateral displacements (and not lateral forces) that cause structures to collapse under their own weight during the earthquake. However, force-based seismic design is well-established in current seismic design codes, because:

– structural engineers are familiar with force-based design for other types of actions (such as gravity and wind loads),
– static equilibrium for a set of prescribed external loads is a robust basis for the analysis, and
– tools for the direct verification of structures for seismic deformations are not considered yet as fully developed for practical application.

The last bullet point refers both to nonlinear analysis methods for the calculation of deformation demands and to the estimation of deformation capacities of structural members.

For all these reasons, it seems that in the foreseeable future force-based seismic design for energy dissipation and ductility will not disappear from design codes and practice.

Force-based seismic design for ductility is based on the inelastic response spectrum of a single-degree-of-freedom (SDOF) system with elastic-perfectly plastic force-displacement curve, F-δ, in monotonic loading. For a fixed value of viscous damping (the value $\zeta = 5\%$ is commonly adopted by convention), the inelastic spectrum relates:

– the period, T, of the SDOF system;
– the ratio $q = F_{el}/F_y$ of the peak force, F_{el}, that would had developed if the SDOF system were linear-elastic, to the yield force of the system, F_y, (q is called "behaviour factor" in Europe, while the term "force reduction factor" or "response modification factor" and the symbol R are used in the US for it) and

– the maximum displacement demand of the inelastic SDOF system, δ_{max}, expressed as a ratio to the yield displacement, δ_y (i.e. as the displacement ductility factor, $\mu_\delta = \delta_{max}/\delta_y$).

Eurocode 8 has adopted the inelastic spectra proposed in (Vidic et al. 1994):

$$\mu_\delta = q, \qquad \text{if } T \geq T_C \tag{1.1}$$

$$\mu_\delta = 1 + (q-1)\frac{T_C}{T}, \quad q = 1 + (\mu_\delta - 1)\frac{T}{T_C} \quad \text{if } T < T_C \tag{1.2}$$

where T_C is the "transition" or "corner" period of the elastic spectrum between the constant spectral pseudo-acceleration and the constant spectral pseudovelocity ranges (see Fig. 1.1, for inelastic spectra normalised to peak ground acceleration of 1 g, with $T_C = 0.6$ s).

The reduction in force response due to ductility bears certain similarities with the effect of higher viscous damping on an elastic SDOF system. The underlying mechanism is similar: energy dissipation; viscous in the case of the elastic SDOF, of hysteretic nature for the elastic-perfectly plastic one. Equation (1.1), applicable in the intermediate-to-long period range, expresses Newmark's well known "equal displacement rule", i.e. the empirical observation that in the constant spectral pseudo-velocity range the peak displacement response of the inelastic and of the elastic SDOF systems are approximately the same. The underlying physical reason is that inertia tends to keep the mass of a flexible SDOF system at the same absolute position while the ground moves underneath, no matter whether the spring of the system yields or not. Equation (1.2) suggests that a very high ductility is needed to appreciably reduce the peak force in a very stiff system (i.e., one with $T << T_C$): for the hysteretic energy dissipation to significantly reduce the force response, the system has to undergo large displacements, which, when divided by the low yield displacement, δ_y, of the very stiff system are translated to very high ductility demands.

The "behaviour factor" q (as well as the "force reduction" or "response modification" factor R) is applied as a global reduction factor of the internal forces that would

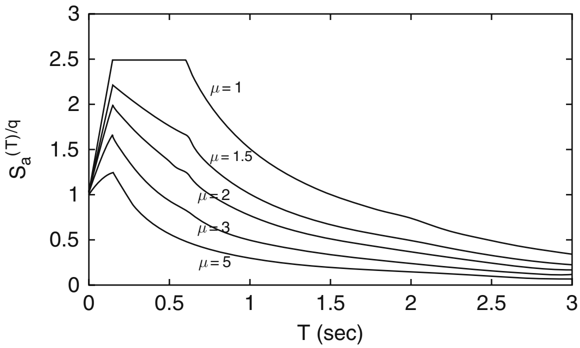

Fig. 1.1 Inelastic spectra from Eqs. (1.1) and (1.2) normalised to peak ground acceleration

develop in the fictitious representation of the structure as elastic with 5% damping (equivalently, on the seismic inertia forces that would develop in the hypothetical elastic structure and induce the seismic internal forces). In this way the seismic internal forces for which the members of the structure are dimensioned can be calculated through linear elastic analysis. A set of prescriptive rules are used, then, to provide the structure with the real aim of the design, namely the capacity to withstand a peak global displacement at least equal to its global yield displacement times the displacement ductility factor, μ_δ, corresponding to the value of q applied for the reduction of elastic force demands (cf. Eqs. (1.1) and (1.2)). The so designed and detailed structure is considered to have "ductility" or "energy-dissipation" capacity – a more general term often used in Europe and in Eurocode 8, as ductility during cyclic response implies that the members and the structure as a whole dissipate a major part of the seismic energy input through hysteresis.

1.2.2 Force-Based Dimensioning of Ductile "Dissipative Zones" and of Other Regions of Members

Not every member or location in a structure is capable of developing ductile behaviour and hysteretic energy dissipation. Typical force-deformation relations (e.g., of moment (M) to curvature (ϕ), or of Force (F) to deflection (δ), etc.) of "ductile" members, regions or mechanisms of load transfer are as those shown in Fig. 1.2(a) for shear span ratio $L_s/h = 2.5$ for monotonic loading or in Fig. 1.2(c) for cyclic loading. It is such members, regions etc., that are entrusted through "capacity design" for inelastic deformations and energy dissipation. Elements, regions or mechanisms of force transfer with force-deformation behaviour as shown for $L_s/h = 1.9$ in Fig. 1.2(a) for monotonic loading or in Fig. 1.2(b) for cyclic loading are "brittle" (or "non-ductile"). They are the ones shielded through "capacity design" from the inelastic action they are incapable of.

Once it yields, a ductile element, etc., can undergo large (sometimes limitless) inelastic deformations at no additional resistance. In concrete, this type of behaviour is characteristic of pure flexure (i.e. without axial load) and of flexural deformations (curvatures, chord rotations, etc.), resembling the behaviour of hinges that allow limitless rotation under zero moment. For this reason regions exhibiting after yielding the behaviour depicted in Fig. 1.2(c) are termed "plastic hinges". They are finite length regions of prismatic concrete members (beams, columns, slender walls) where phenomena like wide cracking, spalling of concrete and yielding and buckling of longitudinal bars are concentrated and where the behaviour accompanying or signaling ultimate conditions (fracture of longitudinal bars, disintegration of concrete, etc.) take place.

The black-and-white distinction of members as "ductile" and "brittle" is convenient. However, the behaviour of the different types of concrete members covers a very broad range from absolute "brittleness" to limitless "ductility". A convenient measure of "ductility" is the available value of the displacement ductility factor of

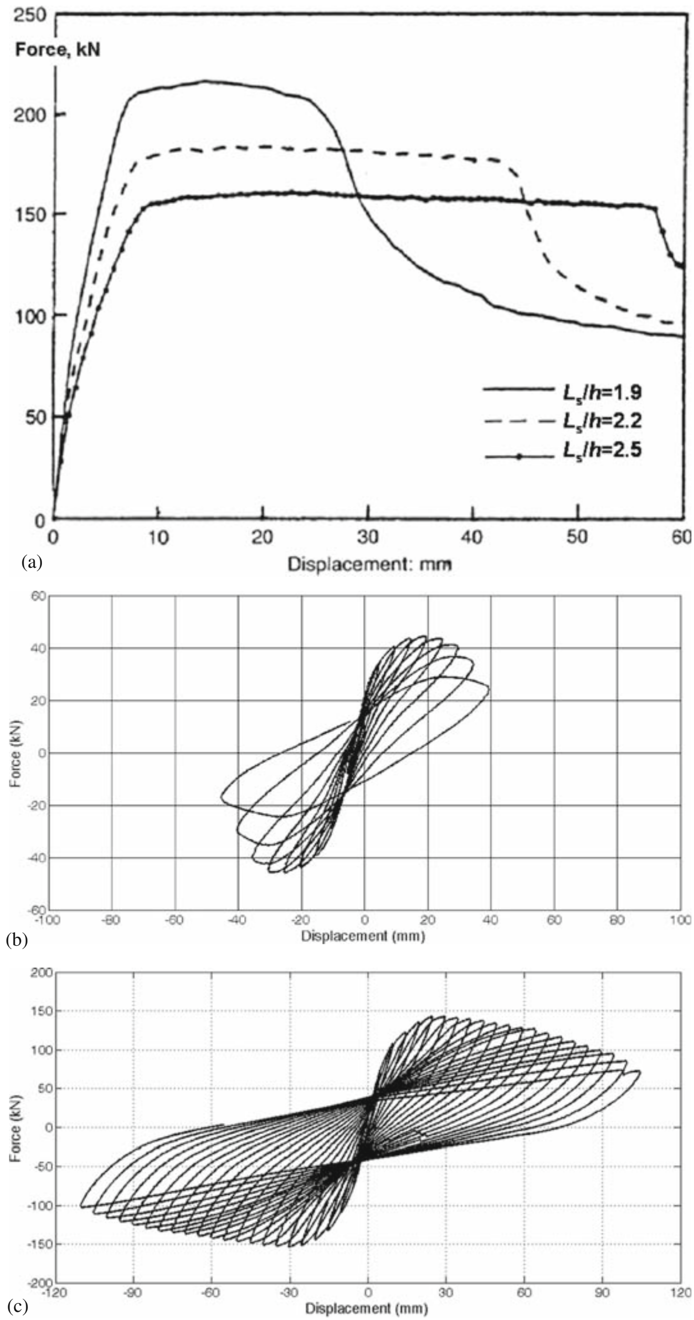

Fig. 1.2 Force-displacement curves for typical: (**a**) ductile behaviour for shear span ratio $L_s/h = 2.5$ to semi-brittle for $L_s/h = 1.9$ in monotonic loading (adapted from Garstka 1993); (**b**), brittle behaviour in cyclic loading (Bousias et al. 2007a); (**c**) ductile behaviour in cyclic loading (Bousias et al. 2007b)

the member, μ_δ (defined as the ratio of its ultimate deflection to the deflection at the corner of a bilinear approximation of the member's force-deflection curve up to ultimate deformation ("yield" deflection)). A conventional limit $\mu_\delta = 2.5$ often distinguishes ductile from brittle behaviour.

To limit occurrence of inelastic deformations only to those members, regions and mechanisms capable of ductile behaviour and hysteretic energy dissipation, while the rest of the structure stays in the elastic range, seismic design codes use a special instrument of seismic design called "Capacity design" and described in detail in Section 1.3. With this instrument a hierarchy of strengths between adjacent structural members or regions, and between different mechanisms of load transfer in the same member is achieved, so that members, regions and mechanisms capable of ductile behaviour and hysteretic energy dissipation are the first ones to develop inelastic deformations. More important, they do so in a way that precludes forever the development of inelastic deformations in any member, region or mechanism deemed incapable of ductile behaviour and hysteretic energy dissipation. Among all current seismic design codes, Eurocode 8 makes the most systematic and extensive use of capacity design to control the inelastic response mechanism (see Section 1.3 for details). Eurocode 8 calls the regions of members which are entrusted for hysteretic energy dissipation "dissipative zones". These regions are designed and detailed to provide the required ductility and energy-dissipation capacity. In concrete, an equivalent term is a "plastic hinge" region or zone, as concrete members can develop hysteretic energy dissipation and ductility only in flexure.

Before designing and detailing a "dissipative zone" for the necessary ductility and energy dissipation capacity, the designer should first dimension it for a force resistance, R_d, at least equal to the action effect, E_d, computed from the elastic analysis for the design seismic action plus the concurrent gravity loads:

$$E_\mathrm{d} \leq R_\mathrm{d} \tag{1.3}$$

The value of E_d in Eq. (1.3) is due to the "design seismic action" (as defined in Sections 1.1.1, 1.1.3 and 1.1.4) and to the quasi-permanent value of the other actions expected to act concurrently. The Eurocode "Basis of Structural Design" (CEN 2002) calls this combination of actions "seismic design situation" (this is the reason for subscript "d" in E_d) and defines the quasi-permanent value of the other actions as the nominal value of permanent loads plus the arbitrary-point-in-time expected value ("quasi-permanent") of gravity loads due to imposed (i.e., live) loads or snow. Normally E_d is calculated through linear analysis. Then the value of E_d may be found by superposition of the seismic action effects from an analysis for the seismic action alone, to the action effects from the analysis for the other actions in the seismic design situation. Second-order effects should be taken into account in the calculation of E_d.

All regions and mechanisms not designated as "dissipative zones" are designed to provide a design value of force resistance, R_d, at least equal to an action effect, E_d, obtained not from the analysis but through "capacity design", as explained in detail in Section 1.3.

The value of force resistance in Eq. (1.3) incorporates one or more safety factors that reduce the nominal value of resistance (i.e., the one calculated using the nominal dimensions of the member and the nominal properties of the materials). In the Eurocodes this is called design value of resistance (hence subscript "d" in R_d). For concrete members the Eurocodes (CEN 2002, 2004b) compute the value of R_d using design values of material strengths: the characteristic or nominal values, f_k (i.e., the nominal yield stress of the reinforcement, f_{yk}, the characteristic 28-day cylindrical compressive strength of concrete, f_{ck}), divided by the corresponding partial factors γ_M for materials. As the γ_Ms are safety elements, they are Nationally Determined Parameters with values specified in the National Annexes to the Eurocodes. Eurocode 8 itself does not recommend the values of γ_M to be used for seismic design. It just mentions in notes the following options:

1. To use the same values of γ_M as in design against monotonic, non-seismic actions (e.g. for the "persistent and transient design situation" in CEN (2002), i.e. the combination of factored permanent actions and factored imposed actions – i.e. live loads – or wind). This option is very convenient for the designer, as he/she may then dimension the dissipative zone to provide a design value of force resistance, R_d, at least equal to the largest among the two action effect due to the "persistent and transient design situation" and that in the "seismic design situation". As for all Nationally Determined Parameters, values of γ_M are specified in the National Annex, in this case that to Eurocode 2. Eurocode 2 itself (CEN 2004b) recommends in a note the following values of γ_M for the "persistent and transient design situation": $\gamma_s = 1.15$ for the strength of the reinforcement, $\gamma_c = 1.5$ for any strength property of concrete.
2. To use the values $\gamma_M = 1$ applicable for design against accidental actions. This is sensible for regions of low to very low seismicity, where knowledge of historical seismicity is not sufficient to support statistical association of the "design seismic action" with a probability of being exceeded in 50 years (or a mean return period). In such cases the "design seismic action" may be conventionally chosen based more on judgement than on a probabilistic hazard analysis. It may have less than 10% probability of being exceeded in 50 years (i.e., mean return period longer than 475 years) and qualify for characterisation as an accidental action. In that case dissipative zones will be dimensioned separately for the action effect due to the persistent and transient design situation, computing the design value of force resistance, R_d, in Eq. (1.3) with $\gamma_M > 1$, and separately for the action effect of the "seismic design situation", using $\gamma_M = 1$ in the calculation of R_d.

Note that the more safe-sided approach 1 above implicitly accounts for some reduction in force resistance due to inelastic cyclic loading (low cycle fatigue). If the actual reduction is large and the value of R_d against monotonic, non-seismic actions is grossly inadequate, a special rule, applicable for inelastic cyclic loading, should be provided by the seismic design code.

1.3 Control of Inelastic Seismic Response Through Capacity Design

1.3.1 The Rationale of Capacity Design

As pointed out in Section 1.2, the horizontal displacement at the point of application of the resultant lateral force due to the design seismic action is known in good approximation, if the fundamental period, T, of the SDOF system is given. Moreover, the maximum energy to be converted to potential (i.e. deformation) energy is also approximately known: the maximum kinetic energy during the response, to be converted to potential energy during the following quarter-cycle, is roughly equal to one-half the total mass times the square of the spectral pseudovelocity, S_v, which for $T \geq T_C$ (cf Eq. (1.1)) is roughly independent of the value of T.

The seismic design of the building determines how the (roughly) given peak global displacement and peak kinetic energy are distributed to the various elements of the building. To distribute them just to those elements best suited to withstand these demands, current seismic design codes use “capacity design” as the main instrument. In the detailed design phase “capacity design” works with and on the strengths of individual elements to ensure that all-along the load path of inertia forces, from the masses to the foundation, the strength of the structural system is governed by the ductile elements. Although capacity design is used during detailed design, its effectiveness depends strongly on the layout and sizing chosen early on, during conceptual design.

The elements to which the peak global displacement and deformation energy demands are channeled by capacity design are selected on the basis of the following criteria:

1. The elements’ “ductility”, i.e., their capacity to develop large inelastic deformations and dissipate energy under cyclic loading, without substantial loss of force-resistance.
2. The importance of the element for the stability of other elements and for the integrity of the whole. Vertical elements are more important than horizontal ones and their importance increases from the roof to the foundation, as their failure may precipitate loss of support for all overlying elements.
3. The accessibility of the element and the difficulty to inspect and repair any damage.

On the basis of the criteria above, a hierarchy of the various elements and regions of the structure can be established, determining the order in which they are allowed to enter the inelastic range during the seismic response. “Capacity design” is used, then, to ensure that this order is indeed respected. As we will see in more detail later in this section, “Capacity design” works as follows:

Once the elements or regions which are more important for the system, or more difficult to inspect/repair, or inherently less “ductile” are identified, “capacity design” determines their required force resistance on the basis of the available force capacities of neighbouring elements or regions which have been ranked as less important, easier to inspect/repair, or inherently more "ductile". The required force resistances of the former elements or regions are determined, so that the latter ones exhaust their force resistances (i.e. yield) before the former and in a way that shields them from yielding. "Capacity design" is based on equilibrium alone, resembling in this respect the static method of plastic design, which gives a lower-bound type of solution.

1.3.2 The Importance of a Stiff and Strong Vertical Spine in a Building

In structures which have horizontal elements at various levels, forming "storeys" (as in multistorey buildings), the spreading of the inelastic deformation demands throughout the structure implies that inelastic action develops in every single storey. For this to be kinematically possible in a concrete building, the beam-column nodes along any column (or any vertical element, in general) should stay on the same line during the seismic response. This implies that vertical elements should:

– stay in the elastic range throughout their height, from the base to the roof, and
– rotate at the base, either by developing a plastic hinge just above the connection to the foundation system, or by rigid-body rotation of their individual foundation element with respect to the ground.

Under these conditions, large horizontal displacements of the storeys are kinematically possible only if plastic hinges form at both ends of every single horizontal member in the system. Such a pattern of plastic hinges and deformations corresponds to the widest possible spreading of the global displacement demand and energy dissipation throughout the entire structural system. It gives, therefore, the smallest possible local deformation and energy dissipation demand on individual members or locations.

In the building of Fig. 1.3(b)–(e) rotations take place at plastic hinges at both beam ends, as well as at plastic hinges at the base of the vertical elements (in Fig. 1.3(b) and (d)) or at the interface between the foundation element and the ground (in Fig. 1.3(c) and (e)). In all these cases, if the intended pattern of distributed plastic hinges forms simultaneously throughout the structure, the maximum chord rotation demand at beam ends or at the base of vertical elements[1] is about equal to the roof displacement, δ, divided by the total building height, H_{tot} (i.e. to the average drift ratio of the building, δ/H_{tot}). Moreover, the demand value of the chord rotation ductility factor at member ends (i.e. the peak chord rotation demand

[1] The chord rotation at a member end is the angle between the normal to the member section there and the chord connecting the two member ends, see Fig. 1.4. If a plastic hinge forms at an end, the plastic part of the chord rotation there is about equal to the plastic hinge rotation.

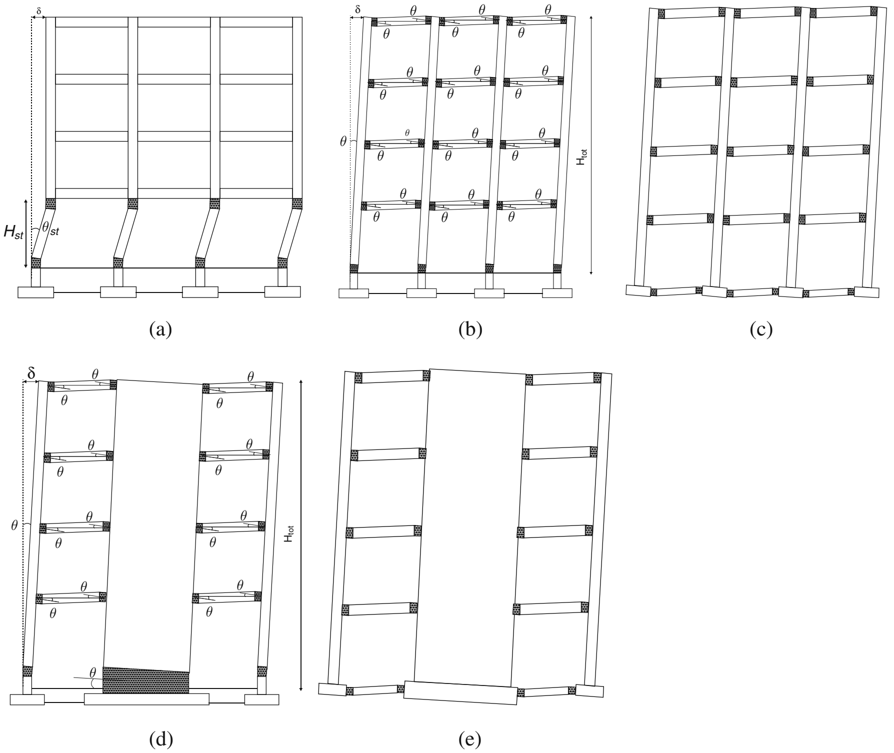

Fig. 1.3 Plastic mechanisms in frame and wall systems: (**a**) soft-storey mechanism in weak column/strong beam frame; (**b**), (**c**) beam-sway mechanisms in strong column/weak beam frame; (**d**), (**e**) beam-sway mechanisms in wall system

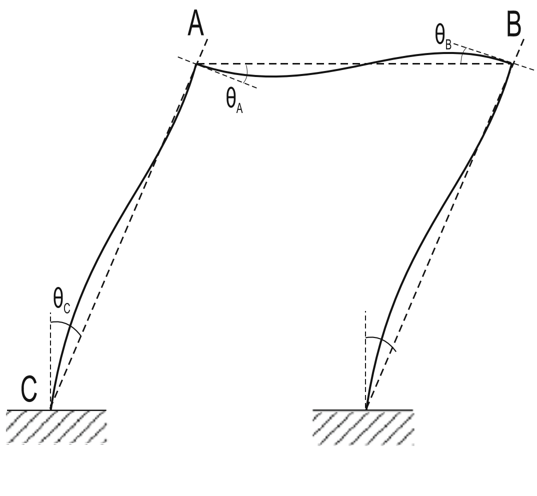

Fig. 1.4 Definition of chord rotation at member ends

during the response, divided by the chord rotation at that end at yielding of the element there) is roughly equal to the demand values of the top displacement ductility factor, μ_δ.[2] According to Eqs. (1.1) and (1.2), μ_δ is about equal to q and assumes relatively low values, well within the capacities of concrete members with appropriately detailed end regions. So, the seismic design of buildings that develop the "beam-sway" mechanisms of Fig. 1.3(b)–(e) is very cost-effective, in the sense that fairly high q-factor values can be relatively easily achieved.

The other extreme is shown in Fig. 1.3(a), where all inelastic deformations take place in a single storey. This is kinematically possible only if all vertical elements in the storey develop plastic hinges at both ends and in opposite bending (i.e. with the same sense of action of bending moments at the two ends). If such a "soft-storey" or "storey-sway" mechanism develops, the chord rotation demand at the ends of the vertical elements of the "soft-storey" are about equal to the roof displacement demand, δ, divided by the height of the soft-storey, h_i. For given value of δ such rotation demands are about equal to those developing in a "beam-sway" mechanism times $H_\mathrm{tot}/h_\mathrm{i}$. By the same token the chord rotation ductility ratio demand in the soft-storey columns is about equal to $H_\mathrm{tot}/h_\mathrm{i}$ times the global displacement ductility factor, μ_δ, derived from the q-factor value used in the design according to Eqs. (1.1) and (1.2). The chord rotation capacities required to meet these demands in medium- or high-rise buildings with $H_\mathrm{tot} >> h_\mathrm{i}$ are not reliably attainable, even with special detailing for very high ductility and energy dissipation capacity. Therefore, it is not feasible to design and detail a building other than a low-rise one or for low-seismicity to develop a "soft-storey" or "storey-sway" mechanism of the type of Fig. 1.3(a).

The best way to spread the global inelastic deformation and energy dissipation demand to the entire structural system and prevent its concentration to a "soft-storey" is by providing a strong and stiff spine consisting of vertical elements that are forced by design to stay elastic above their base. This is achieved by overdesigning these vertical elements relative to the horizontal ones and/or to the internal force demands from the analysis, without any overdesign of the horizontal elements and of the region of the vertical elements at their connection to the foundation. Sections 1.3.4 and 1.3.5 describe how this is pursued through "capacity design" of the columns or walls, respectively.

So far the importance of strong vertical elements for spreading the total deformation and energy dissipation demand to the entire system has been emphasised. Capacity-designing the vertical elements to be strong enough to achieve this end is consistent with the concept of "capacity design" as enforcement of an inelastic response mechanism that does not entail plastic hinging in vertical elements, as these elements are:

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[2] In reality plastic hinges form sequentially, starting at the lower part of the building and never extending throughout their full intended pattern. So the maximum chord rotation and chord rotation ductility factor at any member end will be about double the ideal values of δ/H_tot and μ_δ, respectively.

1. inherently less "ductile" than the beams, due to the adverse effect of axial compression on ductility;
2. more important than the beams, as far as stability and integrity of the whole is concerned.

Modern seismic design codes, such as Eurocode 8, promote development of beam-sway mechanisms in multi-storey buildings thanks to a stiff and strong vertical spine. This is pursued through:

- choices in the structural layout, and
- rules for the dimensioning of vertical members, so that they stay elastic above the base during the response.

More specifically for concrete buildings:

a) Wall systems (or wall-equivalent dual systems according to the definition in Eurocode 8 given in Section 1.4.3.1) are promoted and their walls are (capacity-) designed in flexure and shear to remain elastic above the base.
b) In frame systems and in frame-equivalent dual systems (see Section 1.4.3.1 for the definition of such systems in Eurocode 8) strong columns are directly promoted, through the capacity design of columns in flexure described in Section 1.3.4, so that plastic hinging in columns is prevented. Moreover, in codes that adopt a two-tier seismic design, such as Eurocode 8, strong columns are indirectly promoted by strict interstorey drift limits for the damage limitation seismic action. Unless the columns are large, frame systems cannot easily meet the interstorey drift limits of Eurocode 8 – especially as the cracked stiffness of concrete members is used in the analysis.

1.3.3 Overview of Capacity-Design-Based Seismic Design Procedure

In force-based seismic design using linear analysis with the q-factor, the general seismic design procedure for control of the inelastic response through capacity design is the following:

- Inherently ductile mechanisms of force transfer in "dissipative zones" are dimensioned so that their design resistance, R_d, and the design value of the corresponding action effect from the analysis for the combination of the design seismic action and the concurrent gravity actions, E_d, satisfy Eq. (1.3). In concrete buildings, this phase is normally limited to dimensioning of the end sections of beams in flexure and of the base section of vertical elements (at the connection to the foundation).
- Non-ductile mechanisms of force transfer within or outside the dissipative zones are dimensioned to remain elastic until and beyond yielding of the ductile mechanism(s) of the dissipative zones, through overdesign with respect to the

corresponding action effects from the analysis, E_d. This overdesign is normally accomplished through "capacity design", where the already dimensioned ductile mechanisms of force transfer are assumed to develop overstrength capacities, $\gamma_{Rd}R_d$, and the action effects in the non-ductile mechanisms of force transfer are computed from equilibrium alone.

– Dissipative zones are detailed to provide the deformation and ductility capacity that is consistent with the demands placed on them by the design of the structure for the chosen q-factor value.
– The foundation to the ground is capacity-designed on the basis of the overstrength of ductile mechanisms of force transfer in dissipative zones of the superstructure. Foundation elements are normally capacity-designed as well to stay elastic beyond yielding in dissipative zones of the superstructure. The designer may also use the option to dimension and detail them for energy dissipation and ductility as in the superstructure, despite the difficulty to repair them.

1.3.4 Capacity Design of Columns in Flexure

The objective of current seismic design codes is to force plastic hinges out of the columns of frame systems and into the beams, so that a beam-sway mechanism develops and a soft-storey is prevented. To this end, at beam-column nodes columns are (capacity-)designed to be stronger than the beams, with an overstrength factor of γ_{Rd} applied on the design values of the moment resistances of beams:

$$\Sigma M_{Rc} \geq \gamma_{Rd} \Sigma M_{Rb} \tag{1.4}$$

In Eq. (1.4) M_{Rc} or M_{Rb} denote the moment resistances of columns or beams, respectively. The summation at the left-hand-side extends over the column sections above and below the joint, while the one at the right-hand-side is over all beam ends framing into the joint (Fig. 1.5). Eurocode 8 adopts $\gamma_{Rd} = 1.3$ for the overstrength factor, while US codes (BSSC 2003, SEAOC 1999, ICBO 1997, ACI 2008) use $\gamma_{Rd} = 1.2$.

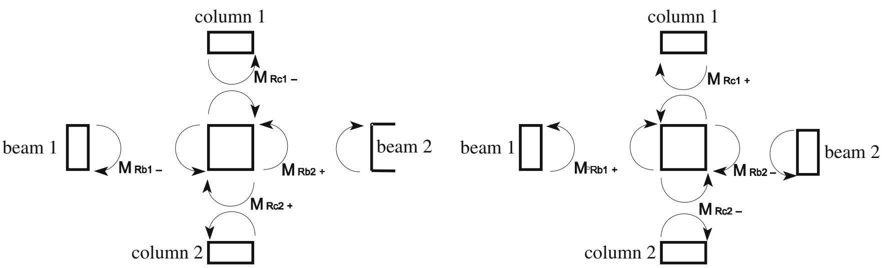


Fig. 1.5 Beam and column moment resistances at a joint, for the implementation of the "Capacity-Design" rule, Eq. (1.4)

Equation (1.4) is verified separately in each one of the two main horizontal directions of the building in plan. For a beam framing into a joint at an angle α to the horizontal direction in which Eq. (1.4) is checked, its M_{Rb}-value enters Eq. (1.4) multiplied times $\cos\alpha$. Equation (1.4) is checked in each horizontal direction, first with both column moment resistances acting on the joint in the positive (clockwise) sense about the normal to that horizontal direction (the direction of the frame) and then in the negative (counterclockwise) sense. Beam moment resistances are always taken to act on the joint in the opposite sense with respect to the column capacities (Fig. 1.5).

Equation (1.4) should normally be checked at the centre of the joint (at the theoretical node at the intersection of the beam and column centrelines), because equilibrium of moments on the joint refers to that point. This would entail transferring the moment resistances from the faces of the joint to the theoretical node: by multiplying the sum of the column moment resistances times $(1 + h_b/H_{cl})$ and that of the beams times $(1 + h_c/L_{cl})$, where h_b, h_c denote the cross-sectional depth of the beam and the column, respectively, in the vertical plane within which Eq. (1.4) is checked, and L_{cl}, H_{cl} are the average clear span of the beams on either side of the joint, or the average clear storey height above and below the joint, respectively. Both Eurocode 8 and the US codes allow using instead in Eq. (1.4) as M_{Rc} and M_{Rb} the moment resistance of the columns and the beams at the face of the joint, respectively. This simplification is normally on the safe side, because in general we have $h_b/H_{cl} \geq h_c/L_{cl}$.

US codes require that the nominal values of M_{Rb} and M_{Rc} (those resulting from the characteristic or nominal values of material strengths, f_{ck}, f_{yk}, instead of the design values, f_{cd}, f_{yd}) be used in Eq. (1.4). For simplification, Eurocode 8 allows using instead the design values of the member moment resistances, $M_{Rd,b}$ and $M_{Rd,c}$ for M_{Rb} and M_{Rc}, respectively. Note that, if the values of material partial factors, γ_M, applicable for non-seismic actions are adopted also for seismic design (option 1 in Section 1.2), the difference between M_{Rd} and the value of M_R for nominal material strengths is larger in the columns than for beams. So, compared to the use of M_{Rc} and M_{Rb} for nominal strengths on both sides of Eq. (1.4), the Eurocode 8 approach gives more safe-sided results for the columns (however, less so than the US approach).

With these differences and the higher value of γ_{Rd} (1.3 versus 1.2), the application of Eq. (1.4) in Eurocode 8 seems to be more safe-sided than in US codes. However, this difference may be overshadowed by how the code accounts for the contribution of slab bars parallel to the beam to the value of M_{Rb} in negative (hogging) bending. There is ample experimental and field evidence that, when the beam is driven past flexural yielding in negative bending and into strain hardening, such reinforcing bars in the slab are fully activated as tension reinforcement of the beam, even when they are at a significant distance from the web. For T-beams (ACI 2008) specifies the total width of the slab effective in tension as 25% of the span, but not larger than 16 times the slab thickness, h_f, plus the web width. For L-beams (ACI 2008) considers that the width of the slab beyond the web which is effective in tension is one-twelfth of the span, but not more than $6h_f$. Eurocode 8 specifies

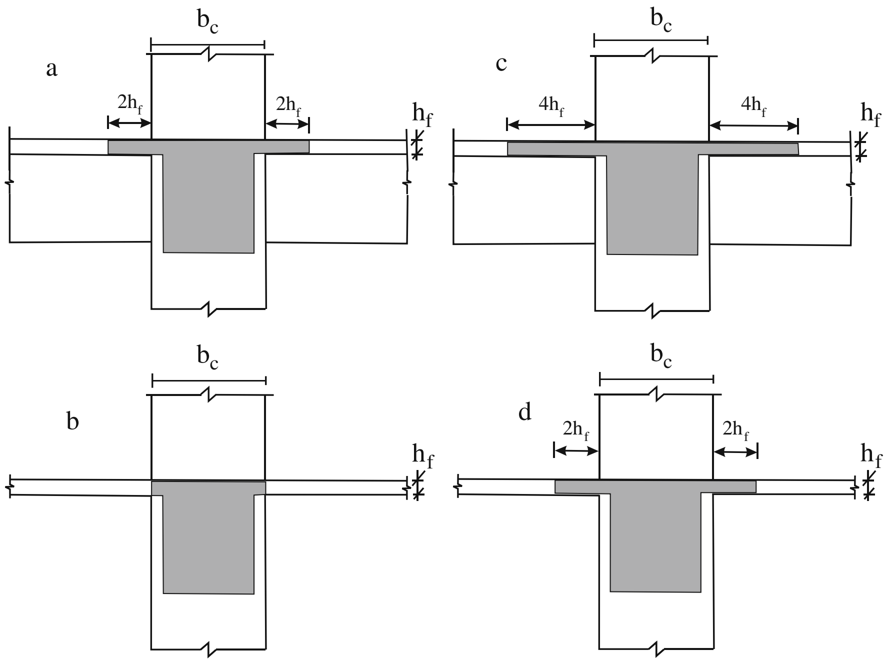


Fig. 1.6 Width of the slab effective as tension flange of beams at the support on a column, according to Eurocode 8 (**a**, **b**: at exterior column; **c**, **d**: at interior column)

as effective in tension a much smaller slab width from the side of the column into which the beam frames, as shown in Fig. 1.6:

- at joints with interior columns within the plane of the frame where Eq. (1.4) is checked:
  - $4h_f$, if a transverse beam of similar size frames into the joint on the side in question, or
  - $2h_f$, if there is no such transverse beam;
- at the two exterior columns within the plane of the frame where Eq. (1.4) is checked:
  - $2h_f$, if a transverse beam of similar size frames into the joint on the side in question, or
  - zero, if there is no such transverse beam.

These effective slab widths are specified in Eurocode 8 for the ULS dimensioning of beams at the supports to columns against the negative (hogging) bending moment from the analysis for the design seismic action combined with the concurrent gravity loads. Slab bars which are parallel to the beam and well anchored within the joint or beyond may count as top beam reinforcement, to reduce the tension reinforcement to be placed within the width of the web. In that context, the effective in tension width of the slab on each side of the web has been chosen in Eurocode 8 lower than the values of about 25% of the beam span suggested by field and experimental evidence, to be safe-sided for the dimensioning of beam top bars. However, this leads to underestimation of $M_{\mathrm{Rd,b}}$ for negative bending and hence is on the unsafe side as far as prevention of column hinging through fulfillment of Eq. (1.4) is concerned (see e.g., (Panagiotakos and Fardis 1998)).

Yielding in opposite bending and plastic hinging at both the top and bottom sections of a concrete wall in a storey is extremely unlikely, even for walls with minimum dimensions (e.g. just 0.2 m by 0.8 m). So, when walls provide most of the lateral force resistance (i.e., in wall systems and in wall-equivalent dual systems according to the Eurocode 8 classification of systems, see Section 1.4.3.1) in a horizontal direction of the building, they can normally be trusted for the prevention of a soft-storey mechanism in that direction. So, Eurocode 8 exempts the columns of wall systems or wall-equivalent dual systems from fulfilling Eq. (1.4) in that horizontal direction. Besides, Eurocode 8 does not require meeting Eq. (1.4) in the following cases of columns of frame systems or of frame-equivalent dual systems (see Section 1.4.3.1 for the definition of these systems):

– Around the joints of the top floor. As a matter of fact, it does not make any difference for the plastic mechanism whether the plastic hinge forms at the top of the top storey column or at the ends of the top floor beams. Moreover, columns of the top floor have low axial load, hence good ductility, and are less critical for the stability of the whole than the columns of lower floors. After all, it is difficult to satisfy Eq. (1.4) there, as only one column contributes to the left-hand-side.
– In two-storey buildings, provided that in none of the ground storey columns the axial load in any of the combinations of the design seismic action with the simultaneous gravity loads exceeds 30% of the cross-sectional area times the design value of the concrete compressive strength, f_{cd}. Columns with such a low axial load ratio have good ductility and develop low 2nd-order (P-Δ) effects. So, if a soft-storey mechanism develops at the ground storey of a two-storey building these columns can withstand a displacement ductility demand of about twice the displacement ductility factor, μ_δ, corresponding to the value of q used in design.
– One-out-of-four columns of plane frames with columns of similar size. The designer may choose to skip fulfilment of Eq. (1.4) at an interior column rather than at an exterior one, as only one beam frames into exterior joints and it is easier to satisfy Eq. (1.4) there.

At all column ends where Eq. (1.4) is not checked owing to the exemptions above (including the columns of wall systems or wall-equivalent dual ones), as well as at the base of columns where a plastic hinge will form anyway, the Eurocode 8 detailing rules provide a column ductility supply sufficient for development of a plastic hinge there.

US standards require meeting Eq. (1.4) at every column of frames of the high ductility class, termed "Special Moment Frames". If Eq. (1.4) is not satisfied at a single level of a column of such a frame, the contribution of that column to the frame's lateral strength and stiffness is neglected and the column is dimensioned for gravity loads alone. However, all the requirements for minimum longitudinal and transverse reinforcement of "Special Moment Frames" should be fulfilled all along that column, to sustain the ductility demands imposed on the column by the lateral-force-resisting system, whose lateral displacements it shares.

Fulfillment of Eq. (1.4) just ensures that the column cross-sections above and below the joint will not yield at the same time under uniaxial bending moments having the same sense of action on the joint. Note that, as vertical reinforcement of a column continues into the storey above and covers the column sections both above and below the joint, these sections have about the same moment resistance if they have the same size. Then $M_{\mathrm{Rc}} \approx 0.5\gamma_{\mathrm{Rd}}\Sigma M_{\mathrm{Rb}}$. However, the bending moments that develop above and below the joint during the dynamic response, may have quite different magnitude $M_{\mathrm{E1}} \neq M_{\mathrm{E2}}$. Therefore, the largest of them, e.g. M_{E1}, may possibly reach the corresponding moment resistance, $M_{\mathrm{Rc1}} \approx 0.5\gamma_{\mathrm{Rd}}\Sigma M_{\mathrm{Rb}}$, and cause column yielding. If this happens simultaneously at both top and bottom of all columns in a storey, then a "soft-storey" may develop there momentarily. If Eq. (1.4) is fulfilled, this event will be of very short duration. Although this possibility cannot be ruled out, the inelastic deformations it induces to the storey columns will not be of such magnitude to jeopardise the stability of the whole (Panagiotakos and Fardis 1998).

In closing this general presentation of capacity design of columns in flexure, it is worth noting, again, that Eq. (1.4) is based on equilibrium and the "static method" of plastic design. The relative stiffness of members is not considered, although its impact on the effectiveness of Eq. (1.4) may sometimes be important. More specifically, as the effective stiffness of concrete members is roughly proportional to their moment resistance, columns whose strength is increased relative to the beams to satisfy Eq. (1.4) in all likelihood will also be stiffer than the beams. They will tend to behave then more as vertical cantilevers – i.e. as walls – rather than as columns of a frame. Wall-like columns may develop bending moments with the same sign (i.e. opposite sense of action on the joint) above and below a joint (cf. the bending moment of diagram typical of walls in Fig. 1.7). Then one of the column sections above or below the joint works with the beams against the other section (instead of with it, against the beams) and might force it to yield. Even in such an unlikely event, a "soft-storey" will not develop, because it requires simultaneous yielding at the top and bottom of the vertical elements of the same storey, which is not physically possible if the bending moment diagram has the same sign within the storey, as in Fig. 1.7. Therefore, although the collateral effects of Eq. (1.4) on member stiffness may render Eq. (1.4) meaningless, the end result is the same: no soft-storey can physically develop.

1.3.5 Design of Ductile Walls in Flexure

What essentially distinguishes walls from columns is that walls have much larger stiffness than the beams they may be connected to. As a result, the beams work with the walls mainly as part of the horizontal diaphragms transferring lateral forces to

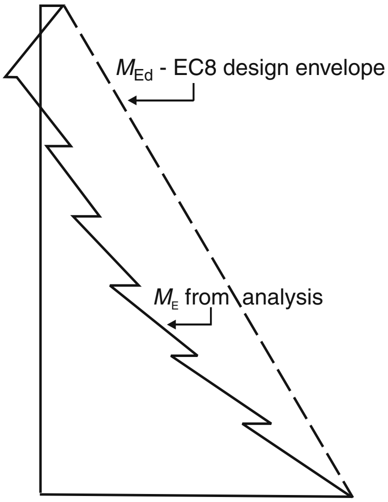


Fig. 1.7 Typical bending moment diagram in a ductile RC wall from the analysis and linear envelope for its design according to Eurocode 8

the walls, rather than as horizontal elements of a frame. Therefore bending moment diagrams that develop during the seismic response in the walls resemble that of a vertical cantilever under horizontal loading (see Fig. 1.7). Notably, the sign of bending moment does not change within a storey (with the possible exception of one or more storeys near the top), while moments decrease considerably from the base to the top of the wall (much more than shear forces do). Moreover, the bending moment at the wall section right above a floor level is normally larger than just below it. As these two sections are crossed by the same vertical bars and an increase in axial compression increases the wall moment resistance, a plastic hinge can conceivably form only at one of these two sections, namely above the floor level. Multiple plastic hinging along the height of the wall may well develop, if the flexural resistance of wall sections at floor levels and at the base (i.e. at the connection to the foundation) are tailored to the elastic seismic moment demands. Even then, a soft-storey mechanism cannot form, as it requires plastic hinging in opposite bending at two different locations along the height of the wall.

To ensure that a wall works as a strong and stiff vertical spine, mobilising all beams into inelastic action and minimising local rotation and ductility demands for given global displacement demand, Eurocode 8 takes measures to localise wall inelastic deformations at its base. A so-designed wall is called “ductile wall”. It is designed and detailed to dissipate energy in a flexural plastic hinge only at the base and remain elastic throughout the rest of its height, in order to promote – or even enforce – a beam-sway mechanism. For a flexural plastic hinge with high ductility and dissipation capacity to develop at its base, a “ductile wall” should be fixed there to prevent relative rotation of the base with respect to the rest of the structural system. Besides, just above its base a ductile wall should be free of openings or large perforations that might jeopardise the ductility of the plastic hinge. To force the wall to stay elastic above the plastic hinge region, Eurocode 8 (but not US standards) requires dimensioning in bending of the rest of the wall height for a linear envelope of the positive and negative wall moments from the analysis for the design seismic action (Fig. 1.7). The envelope intends to cover also a potential increase in bending moments above the base due to higher mode inelastic response after development of the plastic hinge at the base. The rest of the wall does not need to be detailed for high flexural ductility and the design of the wall may be simpler and possibly more economical. Moreover, the rest of the wall above the plastic hinge at the base may be dimensioned in shear disregarding the degradation of cyclic shear resistance in regions that have already yielded in flexure (cf. Section 3.2.4).

US standards (BSSC 2003, SEAOC 1999, ICBO 1997, ACI 2008) do not require designing a wall above the base for flexural overstrength with respect to the demands from the analysis. They rely only on the wall large stiffness and on the fulfilment of Eq. (1.4) by the columns of the system for the prevention of a soft-storey mechanism.

Section 5.6 describes an alternative to "ductile walls", termed "(systems of) large lightly reinforced walls", provided by Eurocode 8 alone among all international seismic design codes. In them flexural overstrength over the seismic demands of the analysis is intentionally avoided anywhere along the height of the wall. This promotes development of plastic hinges in the wall at as many floor levels above the base as physically possible. In this way a given global displacement demand is spread to rotation demands at several locations up the height of the wall. The inelastic deformation demands that need to be resisted by a single location, e.g., at the wall base, are then reduced, facilitating therefore detailing of that location for ductility.

1.3.6 Capacity Design of Members Against Pre-emptive Shear Failure

1.3.6.1 The Principle

Among the two constituents materials of RC members, reinforcing steel is inherently ductile – and as a matter of fact only in tension, as bars in compression may buckle, shedding their compressive force and risking fracture. Concrete is brittle, unless its lateral expansion is well restrained by confinement.

Flexure is the only mechanism of force transfer that allows using to advantage and reliably the fundamental ductility of tension reinforcement and effectively enhancing the ductility of concrete and of the compression steel through lateral restraint. Even under cyclic loading, flexure creates stresses and strains in a single and well-defined direction (parallel to the member axis) and therefore lends itself to the effective use of the reinforcing bars, both to take up directly the tension, as well as to restrain concrete and compression steel (against buckling) transverse to their compressive stresses.

An inelastic stress field dominated by shear is two-dimensional. It induces principal stresses and strains in any inclined direction (especially for cyclic loading) and does not lend itself to effective inelastic action in the reinforcement for the control of the extent of cracking (which, if not effectively controlled, may extend into the compression zone and completely destroy it) and for confinement of concrete. Moreover, after tensile yielding of the transverse bars shear deformations are associated with slippage along wide-open inclined cracks and dissipate very little energy. Last but not least, large reversals of the shear force may accumulate inelastic strains in the same transverse bars crossing both sets of diagonal cracks, leading to uncontrolled crack opening. So, unlike steel members where shear is a ductile force transfer mechanism (as the ductility of steel is always available along the rotating direction of principal strains), in concrete shear is considered brittle and is constrained by design in the elastic range of behaviour. Energy dissipation and cyclic ductility is entrusted only to flexure, in the "plastic hinges" that develop at member ends where seismic bending moments attain their maximum values. The plastic hinge regions are then detailed for the inelastic deformation demands expected to develop there under the design seismic action.

In concrete members the mechanisms of force transfer by shear or by flexure act in series, as both of them have to transfer the same force and ultimate strength is controlled by the weakest of the two mechanisms. So, the shear force transfer mechanism can be constrained to the elastic range through "capacity design". Namely, by dimensioning a concrete member in shear not for the force demand from the analysis but for the maximum shear force that may physically develop in it, as controlled by attainment of the force resistance in flexure. The maximum value of the shear force is computed by:

- expressing (through equilibrium) the shear force in terms of the bending moments at the nearest sections where plastic hinges may form, and
- setting these bending moments equal to the corresponding moment resistances.

Because the bending moment in a plastic hinge cannot physically exceed its moment capacity – including the effect of strain hardening – the so-computed shear force is the maximum possible. Once dimensioned for this design force, a member will remain elastic in shear until and after development of plastic hinges at the sections that affect the value of the shear force.

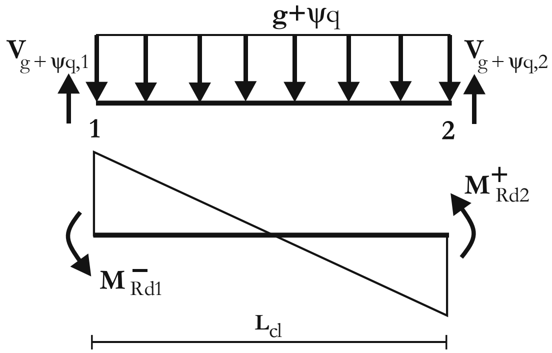
1.3.6.2 Capacity Design Shear of Beams

The basic concept behind capacity design of beams is presented with reference to Fig. 1.8. If the sense of internal forces at beam ends in that figure is considered as positive, equilibrium of moments about one end gives the value of the moment at the other end:

$$V_1 = V_{\mathrm{g}+\psi \mathrm{q},1} + \frac{M_2 + M_1}{L_{cl}} \tag{1.5}$$

$$V_2 = V_{\mathrm{g}+\psi \mathrm{q},2} - \frac{M_1 + M_2}{L_{cl}} \tag{1.6}$$

Fig. 1.8 Equilibrium of forces and moments on a beam



where $V_{g+\psi q,1}$ and $V_{g+\psi q,2}$ are moments of the transverse load acting between the two ends with respect to 2 or 1, respectively, divided by the clear span of the beam, L_{cl} (i.e., they are the reactions to this load when the beam is simply supported).

The maximum value of V_1 develops when the sum $M_2 + M_1$ is maximum, i.e. when both M_1 and M_2 attain their maximum possible positive value. When M_2 and M_1 attain their absolutely maximum possible negative values V_2 reaches its maximum possible value.

If the beam frames at both ends into stronger columns that satisfy there Eq. (1.4) with $\gamma_{Rd} = 1$ the maximum possible positive values of M_1 and M_2 are equal to the corresponding moment resistances. For convenience, these capacities may be taken equal to their design values, M_{Rd}, times an overstrength factor, $\gamma_{Rd} \geq 1.0$. Accordingly, in Eq. (1.5) we may take $M_2 = \gamma_{Rd}M^{+}_{Rd,b2}$, $M_1 = \gamma_{Rd}M^{-}_{Rd,b1}$, while in Eq. (1.6) we have $M_1 = -\gamma_{Rd}M^{+}_{Rd,b1}$, $M_2 = -\gamma_{Rd}M^{-}_{Rd,b2}$. This gives finally the maximum possible ("capacity design") shear forces at the two ends:

$$V_{CD,1} = V_{g+\psi q,1} + \gamma_{Rd}\frac{M^{-}_{Rd,b1} + M^{+}_{Rd,b2}}{l_{cl}} \tag{1.7}$$

$$V_{CD,2} = V_{g+\psi q,2} + \gamma_{Rd}\frac{M^{+}_{Rd,b1} + M^{-}_{Rd,b2}}{l_{cl}} \tag{1.8}$$

Strong beams framing into weak columns (i.e. not satisfying Eq. (1.4) with $\gamma_{Rd} = 1$) are unlikely to develop first plastic hinges at the ends, before the columns do. Assuming that at end i (= 1 or 2) of the beam in question the beam moment is negative and the sum of beam design moment resistances around the joint exceeds that of the columns in the sense associated with negative beam moment at that end (i.e. if $(\Sigma M_{Rd,b})_{i-} > (\Sigma M_{Rd,c})_{i-}$, where subscripts denote the end of the beam and the sign of beam moment there), $M^{-}_{Rd,bi}$ in Eq. (1.6) should be replaced with the beam moment at hinging of the column both above and below the joint at end i . Assuming that the moment input from the yielding columns to the elastic beams is shared by the two beams framing into the joint in proportion to their own moment resistance, the beam moment at end i at the time the columns yield can be assumed equal to the design value of the moment resistance of the beam at that end, $M^-_{\mathrm{Rd,bi}}$, times $[\Sigma M_{\mathrm{Rd,c}}/\Sigma M_{\mathrm{Rd,b}}]_{\mathrm{i}}$, where $\Sigma M_{\mathrm{Rd,b}}$ refers to the sections of the beam across the joint at end i and $\Sigma M_{\mathrm{Rd,c}}$ to those of the column above and below it. Similarly for the positive sense of bending of the beam at end i. So, a rational generalisation of Eqs. (1.7) and (1.8) for the design value of the maximum shear at a section x in the part of the beam closer to end i is (see Fig. 1.9):

$$\max V_{i,d}(x) = \frac{\gamma_{Rd}\left[M^-_{Rd,bi}\min\left(1;\dfrac{\Sigma M_{Rd,c}}{\Sigma M_{Rd,b}}\right)_i + M^+_{Rd,bj}\min\left(1;\dfrac{\Sigma M_{Rd,c}}{\Sigma M_{Rd,b}}\right)_j\right]}{L_{cl}} + V_{g+\psi q,o}(x) \tag{1.9a}$$

In Eq. (1.9a) j denotes the other end of the beam (i.e. if $i = 1$, then $j = 2$). All moments and shears in Eq. (1.9a) have positive sign. The sense of action of $(\Sigma M_{\mathrm{Rd,b}})_{\mathrm{i}}$ on the joint is the same as that of $M_{\mathrm{Rd,bi}}$, while that of $(\Sigma M_{\mathrm{Rd,c}})_{\mathrm{i}}$ is opposite. $V_{\mathrm{g}+\psi\mathrm{q,o}}(x)$ is the shear force at cross-section x due to the quasi-permanent gravity loads, $g + \psi q$, concurrent with the design seismic action, with the beam considered as simply supported (index: o). The value of $V_{\mathrm{g}+\psi\mathrm{q,o}}(x)$ may be conveniently computed (especially if the loads $g + \psi q$ are not uniformly distributed along the length of the beam) from the results of the analysis of the structure for the gravity load $g + \psi q$ alone: $V_{\mathrm{g}+\psi\mathrm{q,o}}(x)$ may be taken equal to the shear force $V_{\mathrm{g}+\psi\mathrm{q}}(x)$ at cross-section x in the full structure, corrected for the shear force $(M_{\mathrm{g}+\psi\mathrm{q,1}} - M_{\mathrm{g}+\psi\mathrm{q,2}})/L_{\mathrm{cl}}$ due to the bending moments $M_{\mathrm{g}+\psi\mathrm{q,1}}$ and $M_{\mathrm{g}+\psi\mathrm{q,2}}$ at the end sections 1 and 2 of the beam in the full structure.

Eurocode 8 adopts Eq. (1.9a) for the capacity design shear of beams, with factor γ_{Rd} accounting for possible overstrength due to steel strain hardening and taken equal to $\gamma_{\mathrm{Rd}} = 1.2$ for beams of Ductility Class High and to $\gamma_{\mathrm{Rd}} = 1$ for those of Ductility Class Medium (see Section 1.4.2.1 for the definition of these Ductility

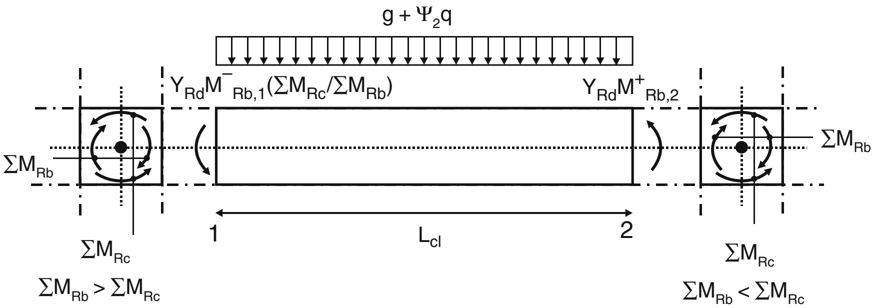


Fig. 1.9 Derivation of the capacity-design shear force in beams according to Eurocode 8

Classes). US codes (BSSC 2003, SEAOC 1999, ICBO 1997, ACI 2008) discount the possibility of column hinging and compute the first term of Eq. (1.9a) with the terms min(...) taken equal to 1.0. They use $\gamma_{\mathrm{Rd}} = 1/0.9 = 1.11$ for the beams of "Intermediate Moment Frames" and $\gamma_{\mathrm{Rd}} = 1.25/0.9 = 1.39$ for those of "Special Moment Frames". For "Intermediate Moment Frames" they allow capping the 1st term in Eq. (1.9a) at twice the shear force at x due to the design seismic action from linear analysis, $V_{\mathrm{E}}(\mathrm{x})$.

With $V_{\mathrm{g}+\psi\mathrm{q,o}}(x)$ taken positive at sections x in the part of the beam closer to end i, the minimum shear in that section is:

$$\min V_{i,d}(x) = -\frac{\gamma_{Rd}\left[M^{+}_{Rd,bi}\min\left(1;\frac{\Sigma M_{Rd,c}}{\Sigma M_{Rd,b}}\right)_i + M^{-}_{Rd,bj}\min\left(1;\frac{\Sigma M_{Rd,c}}{\Sigma M_{Rd,b}}\right)_j\right]}{L_{cl}} + V_{g+\psi q,o}(x) \tag{1.9b}$$

The moments and shears at the right-hand-side of Eq. (1.9b) being positive, its outcome may be positive or negative. If it is positive, the shear at section x will not change sense of action at any time during the seismic response. If it is negative, the shear force does change sense. As described in Section 5.5.2, the ratio:

$$\zeta_i = \frac{\min V_{\mathrm{i,d}}(x_i)}{\max V_{\mathrm{i,d}}(x_i)} \tag{1.10}$$

is used by Eurocode 8 as a measure of the reversal of the shear force at end i, for the dimensioning of the shear reinforcement of beams in buildings of the High Ductility Class (similarly at end j).

The values of $\Sigma M_{\mathrm{Rd,ci}}$ and $\Sigma M_{\mathrm{Rd,cj}}$ to be used in Eqs. (1.9) should be the ones giving the largest absolute value of the capacity design shear in Eq. (1.9a) and the algebraically minimum value of the ζ-ratio in Eq. (1.10). These are the maximum values of $\Sigma M_{\mathrm{Rd,ci}}$ and $\Sigma M_{\mathrm{Rd,cj}}$ within the range of fluctuation of the column axial load from the analysis for the combination of quasi-permanent gravity loads and of the design seismic action. More detailed guidance is given in Section 5.7.3.5.

A positive plastic hinge may develop not at end j of the beam but elsewhere along its span, namely at the point where the available moment resistance in positive bending is first exhausted by the demand seismic moment under the combination of (a) the quasi-permanent gravity loads, $g + \psi q$, and (b) the seismic action that causes beam or column yielding – whichever occurs first – around the joint at end i of the beam. Although the distance between these two likely plastic hinge locations is less than the clear span L_{cl} of the beam, a lower shear force will normally result then near end i of the beam than the value from Eq. (1.9a).

1.3.6.3 Capacity-Design Shear of Columns

The simplest way to derive the capacity design shear of a column is to assume that its ends, 1 and 2, both develop plastic hinges in opposite bending (+ or –) and compute the resulting shear force from equilibrium. Normally no intermediate transverse loads act on columns. So, the capacity design shear is constant throughout the column height and equal to:

$$V_{CD}^{-} = \gamma_{Rd}\frac{M_{Rd,c1}^{-} + M_{Rd,c2}^{+}}{H_{cl}} \tag{1.11a}$$

$$V_{CD}^{+} = \gamma_{Rd}\frac{M_{Rd,c1}^{+} + M_{Rd,c2}^{-}}{H_{cl}} \tag{1.11b}$$

Factor γ_{Rd} in Eqs. (1.11) accounts again for possible overstrength due to steel strain hardening; H_{cl} is the clear height of the column within the plane of bending (in general equal to the distance between the top of the beam or slab at the base of the column and the soffit of the beam at the top).

Normally the column shear capacity is independent of the direction of the shear force. Then only the maximum of the shear forces V_{CD}^{-} and V_{CD}^{+} in Eqs. (1.11) matters. Moreover, usually the column cross-section is symmetric and $M_{Rd,ci}^{+}$ and $M_{Rd,ci}^{-}$ at end i (= 1, 2) are equal. Then Eqs. (1.11a) and (1.11b) give the same outcome.

As shown in Fig. 1.10, a column may not develop plastic hinges at end i ($i = 1$, 2), if plastic hinges develop there first in the beams framing into the same joint at end i (as is normally the case in columns fulfilling Eq. (1.4)). If that happens, the sum of column moments above and below the joint is equal to the total moment resistance of the beam across that joint, $\Sigma M_{Rd,b}$. It may be assumed that this sum is shared by the two column sections above and below the joint in proportion to their own moment resistance. Then, the bending moment at end section i ($i = 1, 2$) of the column may be taken equal to the design value of the moment resistance of the column at that end, $M_{Rd,ci}$, times $(\Sigma M_{Rd,b}/\Sigma M_{Rd,c})_i$, where $\Sigma M_{Rd,b}$ refers to the sections of the beam on opposite sides of the joint at end i and $\Sigma M_{Rd,c}$ to the sections of the column above and below it. The sense of action of $\Sigma M_{Rd,c}$ on the joint is the same as that of $M_{Rd,ci}$, while that of $\Sigma M_{Rd,b}$ is opposite. So, a rational generalisation of Eqs. (1.11) for the design value of the maximum shear of the column is:

$$V_{CD,c} = \frac{\gamma_{Rd}\left[M_{Rd,c1}\min\left(1;\dfrac{\Sigma M_{Rd,b}}{\Sigma M_{Rd,c}}\right)_1 + M_{Rd,c2}\min\left(1;\dfrac{\Sigma M_{Rd,b}}{\Sigma M_{Rd,c}}\right)_2\right]}{H_{cl}} \tag{1.12}$$

Equation (1.12) is the form of capacity design shear of columns adopted in Eurocode 8, with factor γ_{Rd} taken equal to $\gamma_{Rd} = 1.3$ for columns of buildings of Ductility Class High and to $\gamma_{Rd} = 1.1$ for those of Ductility Class Medium (see Section 1.4.2.1 for the definition of these Ductility Classes).

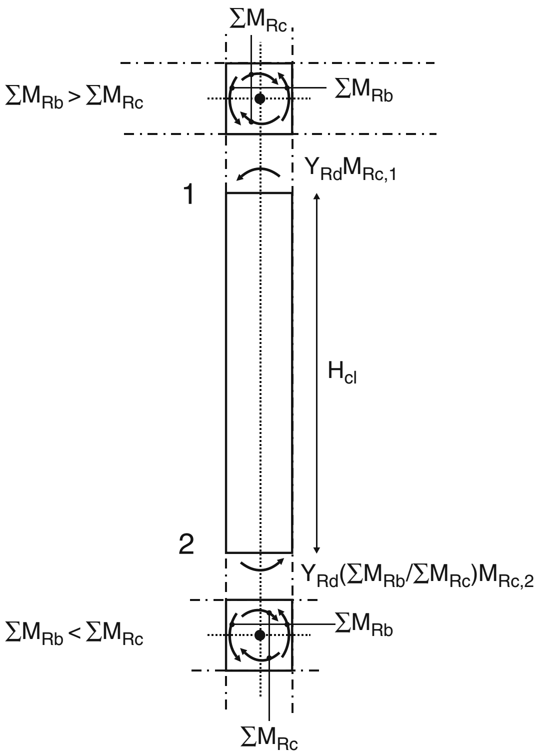


Fig. 1.10 Derivation of the capacity-design shear force in columns according to Eurocode 8

ACI (2008) adopts a format similar to Eq. (1.12) for columns of "Special Moment Frames":

$$V_{CD} = \gamma_{Rd} \frac{\left[\min\left(M_{Rd,c};\ \frac{\min(\Sigma M_{Rd,c},\ \Sigma M_{Rd,b})}{\Sigma\,|M_{Ec}|}|M_{Ec}|\right]_1 + \left[\min\left(M_{Rd,c};\ \frac{\min(\Sigma M_{Rd,c},\ \Sigma M_{Rd,b})}{\Sigma\,|M_{Ec}|}|M_{Ec}|\right]_2}{H_{cl}} \tag{1.13}$$

In Eq. (1.13) the moment input from the yielding elements around the joint at end i (= 1, 2) is shared by the two columns framing into that joint in proportion to their end moments from the analysis for the design seismic action, M_{Ec}; factor γ_{Rd} is taken equal to $\gamma_{\mathrm{Rd}} = 1.25/0.7 = 1.79$. For the columns of "Intermediate Moment RC Frames" (ACI 2008) does not take into account the possibility of beam hinging and uses the simpler version, Eqs. (1.11) with $\gamma_{\mathrm{Rd}} = 1/0.7 = 1.43$. It also caps the value of the capacity design shear to twice the shear force due to the design seismic action from linear analysis, V_{E}.

The values of $M_{\mathrm{Rd,c1}}$ and $M_{\mathrm{Rd,c2}}$ to be used in Eqs. (1.11), (1.12) and (1.13) should be the most adverse ones within the range of fluctuation of the column axial force under the combination of quasi-permanent gravity loads and the design seismic action. If the dependence of the column shear capacity on axial force is taken into account (in fact shear capacity increases with increasing axial compression), more than one possible axial force values should be considered for the calculation of $M_{\mathrm{Rd,ci}}$ ($i = 1, 2$) in Eqs. (1.11), (1.12) and (1.13), in search of the most critical condition for the shear verification of the column and the dimensioning of its transverse reinforcement. If the shear capacity of the column is taken independent of its axial force, then the values of $M_{\mathrm{Rd,c1}}$ and $M_{\mathrm{Rd,c2}}$ should be the maximum ones within the range of fluctuation of the column axial load from the analysis for the combination of the design seismic action and the concurrent gravity loads. More detailed guidance is provided in Section 5.7.3.5.

1.3.6.4 Capacity-Design Shear of "Ductile Walls"

US standards do not require designing walls for overstrength in shear relative to the demands from the analysis or over the seismic action that induces plastic hinging. In Eurocode 8, by contrast, "ductile walls" are designed to develop a plastic hinge only at the base section and to stay elastic throughout the rest of their height. The value of the moment resistance at the base section of the wall, M_{Rdo}, and equilibrium alone are not sufficient for the derivation of the maximum seismic shears that can develop at various levels of the wall. The reason is that, unlike in the members of Figs. 1.8, 1.9 and 1.10, the (horizontal) forces applied on the wall from the floors are not constant but change during the seismic response. In the face of this difficulty, a first assumption made is that, if M_{Rdo} exceeds the bending moment at the base from the elastic analysis for the design seismic action, M_{Edo}, seismic shears at any level of the wall exceed those from the same elastic analysis in proportion to ($M_{\mathrm{Rdo}}/M_{\mathrm{Edo}}$). So, the shear force from the elastic analysis for the design seismic action, V'_{Ed}, should be multiplied by a capacity-design magnification factor ε proportional to $M_{\mathrm{Rdo}}/M_{\mathrm{Edo}}$.

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = \gamma_{Rd}\left(\frac{M_{Rdo}}{M_{Edo}}\right) \leq q \tag{1.14}$$

Factor γ_{Rd} in Eq. (1.14) is meant to capture the overstrength at the base over the design value of the moment resistance there, M_{Rdo}, e.g. owing to strain hardening of the vertical steel.

Section 1.3.5, dealing with flexural design of ductile walls, has already mentioned the possibility of higher mode response after formation of a plastic hinge at the base, i.e., of the response of a structure with little rotational restraint at plastic hinges that have already formed and are loading along the ascending post-yield branch of their moment-rotation relation. This response may increase also the wall shear forces at the base and higher up, to values well beyond those corresponding to plastic hinging at the base according to the predictions of elastic analysis.

The taller and more slender the wall, the more pronounced are such effects, being almost absent in "squat" walls.

To cover both capacity design in shear, expressed by Eq. (1.14), as well as any inelastic higher mode effects, Eurocode 8 has adopted the following expression for walls with ratio of height to horizontal dimension, $h_w/l_w > 2$ ("slender" walls) of Ductility Class High:

$$\varepsilon = \frac{V_{\mathrm{Ed}}}{V'_{\mathrm{Ed}}} = \sqrt{\left(\gamma_{\mathrm{Rd}}\frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1\left(q\frac{S_a(T_C)}{S_a(T_1)}\right)^2} \leq q \tag{1.15}$$

where:

- the overstrength factor γ_{Rd} is taken equal to $\gamma_{\mathrm{Rd}} = 1.2$;
- $S_a(T_1)$ is the value of the elastic spectral acceleration at the period of the fundamental mode in the horizontal direction closest to that of the wall shear force[3] (see Eqs. (4.2) in Section 4.2.1.2), and
- $S_a(T_C)$ is the spectral acceleration at the corner period, T_C, of the elastic spectrum.

The 2nd term under the square root of Eq. (1.15) has been proposed in Eibl and Keintzel (1988) and Keintzel (1990) to capture the increase of shear forces over the elastic overstrength value represented by the 1st term, owing to higher mode effects in the elastic and the inelastic regime of the response. In modes higher than the first one the ratio of shear force to the bending moment at the base exceeds the corresponding value at the fundamental mode, which is considered to be primarily (if not exclusively) reflected by the results of the elastic analysis. The longer the period T_1 of the fundamental mode with respect to the corner period, T_C, of the elastic spectrum (e.g., for flexible frame-wall systems on stiff soil), the higher is the value of ε, reflecting the more significant influence of higher modes on shears. Note that the correction factor in Eq. (1.15) has been proposed in Eibl and Keintzel (1988) and Keintzel (1990) just for the shear force results of the "lateral force" (equivalent static) procedure of elastic analysis for the design seismic action. "Modal response spectrum" elastic analysis fully captures the effects of higher modes on elastic seismic shears, but fails to do so for the inelastic ones, after formation of a plastic hinge at the base.

The 1st term under the square root of Eq. (1.15) assumes a different value for different individual walls of the building (even for coupled ones), but the 2nd term is the same for all walls in the building, regardless of size and contribution to lateral force resistance. Note also that, by including the behaviour factor q, this 2nd term removes part of the reduction of the elastic response spectrum by q, reflecting the smaller influence of inelasticity on the higher mode response of wall structures.

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[3] Strictly speaking, T_1 is the period of the mode with the largest modal mass in the direction closest to that of the wall shear force.

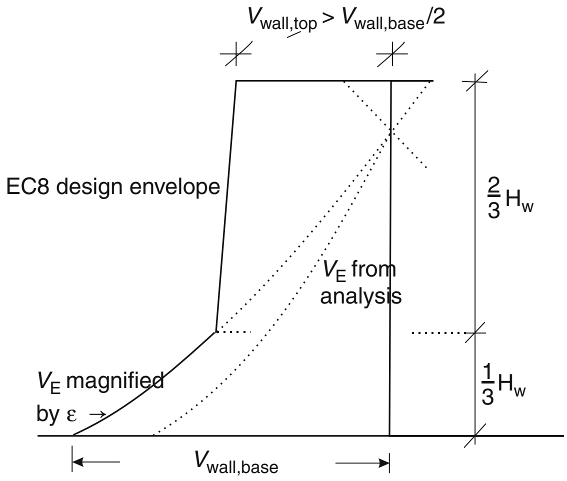

Fig. 1.11 Design envelope of shear forces in "ductile walls" of dual (frame-wall) systems according to Eurocode 8

Equation (1.15) gives safe-sided results, especially when used together with the further increase of shear forces imposed by Eurocode 8 over the upper two-thirds of the height of walls in frame-wall structural systems (see last paragraph of this section and Fig. 1.11).

In walls with ratio of height to horizontal dimension, $h_w/l_w \leq 2$ ("squat" walls) of Ductility Class High Eurocode 8 uses just Eq. (1.14) with $\gamma_{Rd} = 1.2$.

Note that, if the axial force in the wall from the analysis for the design seismic action is high (as, e.g., in slender walls near the corner of high-rise buildings, or in piers of coupled walls), the difference between the algebraically maximum and minimum axial force of the wall in the various combinations of the design seismic action with the concurrent acting gravity loads will be large. The vertical reinforcement at the base of the wall will be governed by the combination of the bending moment from the analysis, M_{Edo}, with the algebraically minimum axial force there (for compression taken as positive), while, under the algebraically maximum axial force the moment resistance, M_{Rdo}, will be much larger than M_{Edo}. Then the value of ε from Eqs. (1.14) and (1.15) may be so high, that the verification of the wall in shear (especially against failure by diagonal compression) may be unfeasible. This may be particularly the case for the piers of coupled walls.

For simplicity, in walls of buildings of Ductility Class Medium Eurocode 8 adopts the following shear magnification factor:

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = 1.5 \tag{1.16}$$

Compared with Eq. (1.15), Eq. (1.16) is much easier to use and gives more economical wall designs in shear. However, this simplicity and economy may be at the detriment of performance, as Eq. (1.16) provides a very low safety margin (if any at all) against flexural overstrength at the base and inelastic higher mode effects.

In walls of Ductility Class High the value of ε from Eqs. (1.14) and (1.15) does not need to be taken greater than the value of the q-factor, so that the final design shear, V_{Ed}, does not exceed the value qV'_{Ed} corresponding to fully elastic response. Of course, ε may not be taken less than the value of 1.5 specified for Ductility Class Medium.

Higher mode effects on inelastic shears are larger at the upper storeys of the wall, and indeed more so in frame-wall structural systems. The frames of such systems restrain the walls in the upper storeys, so that the shear forces at the top storey of the walls obtained from the "lateral force procedure" of elastic analysis are opposite to the total applied seismic shear, turning to positive one or two storeys below the top. Multiplication of these very low storey shears by the factor ε of Eqs. (1.14), (1.15) and (1.16) will not bring their magnitude anywhere close to the relatively high values that may develop there owing to higher modes (cf. dotted curves representing the shear forces from the analysis and their magnified by ε version in Fig. 1.11). Eurocode 8 deals with this question by requiring the minimum design shear at the top of ductile walls of frame-wall systems be at least equal to half the magnified shear at the base, increasing linearly to the magnified value of the shear, $\varepsilon V'_{\mathrm{Ed}}$, at one third of the wall height from the base (Fig. 1.11).

1.4 The Options of Strength or Ductility in Earthquake-Resistant Design

1.4.1 Ductility as an Alternative to Strength

Equations (1.1) and (1.2) show that design seismic forces are about inversely proportional to the demand value of the global displacement ductility factor, μ_δ. So, there is an apparent economic advantage in increasing the available global ductility, to reduce the internal forces for which structural members are dimensioned. Besides the economic one, there are a number of other advantages of ductility as a substitute for strength in earthquake-resistant design:

- If the lateral force resistance of the structure is reduced by dividing the elastic lateral force demands by a high q-factor value, verification of the foundation soil, which by necessity is based on strength rather than on deformation capacity, is much easier.
- A cap on the magnitude of lateral forces that can develop in the structure reduces response accelerations and protects better the contents of the building (including valuable equipment and artefacts), as well as non-structural parts which are sensitive to acceleration (e.g. infill panels in the out-of-plane direction). Note that non-structural elements that are sensitive to deformations (such as infill panels in the in-plane direction) are not adversely affected by inelastic action in

the structural system. The reason is that, according to the "equal displacement rule" expressed by Eq. (1.1) and applying in good approximation for most buildings, lateral displacements and interstorey drifts are equal to those in the elastic structure.
– A structure with ample ductility supply is more resilient to earthquakes much stronger than the design seismic action and less sensitive to the details of the ground motion (i.e., to its frequency content and duration). So, in view of the large uncertainty associated with the extreme earthquake demand in the lifetime of a building, such a structure can be considered as a better earthquake-resistant design. Moreover, it can put its robustness into use against other actions of accidental nature, such as extreme natural or man-made hazards, for which structures are normally not designed.

There are also strong arguments in favour of less ductility and dissipation capacity and more lateral force resistance in seismic design:

– A RC structure that uses its high ductility in a strong earthquake will survive the event, but with large residual deformations, i.e., with significant structural damage, often difficult to repair. In the light of performance-based design and of protection of property as one of its prime motivations, the higher the lateral strength of a structure, the smaller will be the structural damage, not only during more frequent, moderate earthquakes, but also due to the design seismic action and beyond.
– From the construction point of view, detailing of members for ductility normally entails fixing the reinforcement in the form of cages of closely-spaced ties engaging practically every single longitudinal bar, and placing and compacting concrete within and through such cages. So, it is sometimes doubtful that the desired quality of the end product is achieved, even when workmanship is of high level and on-site supervision strict. By contrast, detailing of members just for strength is much easier and simpler.
– Many buildings designed for earthquake resistance possess anyway significant lateral strength, thanks to their force-based design against non-seismic actions. So, they may have significant resistance to earthquake forces, without even been designed for them. Examples include: low-to-medium-rise buildings in low-to-moderate seismicity regions, with gravity loads controlling their design; tall, flexible buildings dominated by wind, etc. In such cases it makes sense to benefit from the available margin of lateral strength, in order to avoid complex and expensive detailing of members for ductility.
– Often the layout of the structural system is unusually complex and irregular and falls outside the framework of the ordinary structural layouts mainly addressed by seismic design standards. In that case the designer may feel more confident for his/her design if he/she narrows the distance between the results of the linear elastic analysis used for dimensioning the members – and the nonlinear seismic response to the design seismic action. This can be achieved through a lower value of the behaviour factor q, implying lower global and local ductility demands.

If the global ductility demand is reduced at the expense of increased lateral strength, application of capacity design may be drastically relaxed, or even omitted. Capacity design rules for columns in bending and beams or columns in shear aim at avoiding overstrength in the ductile modes of behaviour and member failure – e.g., of beams in flexure – with respect to the more brittle ones, notably of all elements in shear. Such overstrengths may occur if the resistance of the more ductile modes is controlled by gravity loads or by minimum reinforcement, while that of more brittle ones is governed by the design seismic action. In structures of low ductility design seismic internal forces are in the order of about two-thirds of those resulting from purely elastic response to the design ground motion. For so high design seismic forces, it is expected that the seismic action will control dimensioning of every single member against all failure modes and there will not be any undesirable overstrengths. Accordingly, capacity design requirements can be waived to simplify the entire design process. Moreover, member ductility demands associated with the low global displacement ductility factor of low ductility structures, are relatively low, even though inelastic deformation demands may not be uniformly distributed throughout the system. Such low local ductility demands can be easily accommodated with detailing for non-earthquake resistant members, which is easier to design for and implement in-situ. So, the selection of a higher or lower ductility level for a structure has very important implications on the design and construction effort. A designer who opts for higher ductility, should have at his/her disposal more advanced design tools along with the experience and expertise necessary for their use, as well as confidence in the construction crews for the implementation of demanding member detailing.

1.4.2 The Trade-Off Between Strength and Ductility – Ductility Classification in Seismic Design Codes

Most modern seismic codes provide more than one combinations of strength and ductility. Some of them let the designer choose the strength-ductility combination, depending on the particular features of the project. Others specify which combination is appropriate, depending on the seismicity of the site, the importance and occupancy of the building and other design parameters.

European or US standards provide a few "discrete" strength-ductility combinations, each one with its own well-defined rules for member dimensioning and detailing. They are, therefore, most convenient for computational implementation and routine application, although they limit significantly the choices available to the designer.

1.4.2.1 Eurocode 8

Eurocode 8 allows trading ductility for strength through the provision of three alternative Ductility Classes (DCs):

- Ductility Class Low (DC L),
- Ductility Class Medium (DC M), or
- Ductility High (DC H).

Buildings of DC L are not designed for any ductility but only for strength. Except certain minimum conditions for the ductility of reinforcing steel, such buildings have to follow only the dimensioning and detailing rules specified in Eurocode 2 (CEN 2004b) for non-seismic actions. They are designed against the earthquake exactly as against other lateral actions, e.g. wind. Although they are expected to respond elastically to the combination of its design seismic action with the concurrent gravity loads, they are entitled to a behaviour factor value of $q = 1.5$ (instead of $q = 1.0$), attributed only to member overstrength over the seismic internal forces they are dimensioned for. The sources of overstrength are:

- The systematic difference between the expected strength of steel or in-situ concrete from the corresponding design values: the mean strength is considered to exceed normally the characteristic value by 8 MPa for concrete, or by about 15% for reinforcing steel. Moreover, in dimensioning the characteristic strengths are divided by the partial factors for materials to arrive at their design values.
- The fact that often the reinforcement is controlled by non-seismic actions and/or minimum reinforcement requirements, etc.
- The use of the same reinforcement at the two cross-sections of a beam or column across a joint, as determined by the maximum required steel area at these two sections.
- The rounding-up of the number and/or diameter of reinforcing bars.

In regions of moderate or high seismicity DC L buildings are, in general, not cost-effective. Moreover, as they do not have any engineered ductility, they may not have a reliable safety margin against an earthquake significantly stronger than the design seismic action. So, they are not considered as suitable for such regions. Eurocode 8 itself recommends using DC L only for "low seismicity cases", for which it is expected to be more economic and easier to apply. It is up to National Authorities, however, to follow this recommendation or not. The definition of what is a "low seismicity case" has also been left to National Authorities. Eurocode 8 recommends that a "low seismicity case" be one where the design ground acceleration on rock, a_g, (including the importance factor of the building, γ_I), does not exceed 0.08 g, or that at the ground surface the site, a_gS is not more than 0.1 g (see Section 4.2.1 for the definitions of a_g and S).

Design of buildings for DC L is allowed by Eurocode 8 in cases beyond those of "low seismicity" when in the horizontal direction considered the value of the seismic design base shear (at the level of the foundation or of the top of a rigid basement) calculated with a behaviour factor of $q = 1.5$ is less than the base shear due to the design wind action, or any other lateral action for which the building is designed using linear elastic analysis.

Design for strength alone without engineered ductility is an extreme, only for special cases. Within the fundamental case of seismic design, namely that of design for ductility and energy dissipation, Eurocode 8 normally gives the option to design for more strength and less ductility or vice-versa, by choosing between Ductility Class Medium (DC M) or High (DC H).

Buildings of DC M or H have q-factors higher than the value of 1.5 considered available thanks to overstrength alone. DC H buildings are entitled higher values of q than DC M ones (see Section 1.4.3.1). They also have to meet more stringent detailing requirements for members (see Tables 5.1, 5.2 and 5.3) and provide higher safety margins in capacity design in shear (see Sections 1.3.6.2 and 1.3.6.3 for the differences in the γ_{Rd} values for the capacity design shear force of DC M and DC H beams or columns, and Section 1.3.6.4 for differences in the shear magnification factor ε for ductile walls). Fardis and Panagiotakos (1997a) have reported on the detailed design of 26 concrete buildings – frame or frame-wall systems – according to the pre-standard version of Eurocode 8 and (Panagiotakos and Fardis 2003, 2004) on the design of nine regular RC frame buildings to the EN-Eurocode 8. The conclusion of both studies was that, although the total quantity of steel and concrete is essentially independent of the Ductility Class adopted for the design, the higher the DC, the larger is the share of transverse reinforcement and of the reinforcement of vertical members in the total quantity of steel. Moreover, DC M and DC H are roughly equivalent in terms of achieved performance under the design seismic action. DC M is slightly easier to design for and implement in-situ and may provide better performance in moderate earthquakes. DC H seems to provide larger safety margins than DC M against local or global collapse under earthquakes (much) stronger than the design seismic action. In high seismicity regions DC H may hold some economic advantage. Its use there will be facilitated by the existing tradition and expertise in seismic design and on-site implementation of complex detailing for ductility.

Eurocode 8 itself does not link selection between the two higher ductility classes to seismicity of the site or importance of the structure, nor puts any limit to their application. It is up to countries to choose for the various parts of its territory and types of construction therein, or – preferably – to leave the choice to the designer, depending on the particular design project.

1.4.2.2 US Standards

US standards (BSSC 2003, ICC 2006) specify the combination of strength and ductility depending on the seismicity of the site, the type of occupancy and the importance of the building. To this end, they introduce "Seismic Design Categories" A–F. A building is classified as A, if the (effective) peak ground acceleration (*EPA*) and the 5%-damped elastic spectral acceleration at 1 s period, S_{a1}, are both below 0.067 g. The next threshold level for *EPA* or S_{a1} is 0.133 g, below which a building is classified as B – or C if it houses an essential or hazardous facility. The next threshold level is 0.2 g, below which a building is classified as C – or D for essential or hazardous facilities. For *EPA* or S_{a1} above 0.2 g of a building is classified as D. If the value of S_{a1} for the MCE (Maximum Considered Earthquake) over firm rock exceeds 0.75 g, a building is classified as F if it houses essential or hazardous facilities, or as E otherwise.

Buildings of “Seismic Design Category” A are only required to have a complete tied-together lateral load resisting system designed for a lateral force of 1% of total weight. “Seismic Design Category” B buildings may just be designed for the seismic internal forces from linear analysis without special detailing, i.e., as “Ordinary Moment Frames” (ACI 2008). “Seismic Design Category” C buildings are subject to mild detailing requirements; concrete frames – but not walls – should satisfy the (ACI 2008) requirements for “Intermediate Moment Frames”. Buildings in “Seismic Design Categories” D, E or F should be detailed for high ductility, with “Special Moment Frames” or walls of “special” ductility, entitled to larger force reduction or response modification factors, *R*, than “Intermediate Moment Frames”.

According to ACI (2008), “Ordinary Moment Frames” are not subject to ductility requirements. “Special Moment Frames” have very good global ductility, thanks to the application of capacity design of columns in bending (see Section 1.3.4) and of beams and columns in shear (see Section 1.3.6). They also have high local ductility, thanks to the application of stringent detailing rules for the longitudinal and transverse reinforcement of all types of members. “Intermediate Moment Frames” do not have to satisfy the capacity design rule of columns in bending, Eq. (1.4), may follow less demanding capacity design of beams and columns in shear (see Section 1.3.6.2) and are subject to less stringent requirements for the longitudinal reinforcement of beams and the transverse bars of columns.

1.4.3 Behaviour Factor q *of Concrete Buildings Designed for Energy Dissipation*

For building structures designed for energy dissipation and ductility, the value of the behaviour factor *q*, by which the elastic spectrum used in linear analysis is divided, depends:

- on the ductility class selected for the design,
- on the type of lateral-force-resisting-system, and
- (in Eurocode 8) on the regularity of the structural system in elevation.

The value of the *q*-factor is linked, indirectly (through the ductility classification) or directly (as in Eurocode 8, see Chapter 5), to the local ductility demands in members and hence to the corresponding detailing requirements.

1.4.3.1 Eurocode 8

The overstrength of materials and elements is presumed to correspond to a *q*-factor value of 1.5, which is assigned to DC L buildings without any association to ductility. This value is also incorporated in the q-factors of buildings of DC M or H. Besides, overstrength of the structural system due to redundancy is explicitly included in the q-factor, through a multiplicative factor α_u/α_1. This is the ratio of the seismic action that causes development of a full plastic mechanism, to the seismic action at formation of the first plastic hinge in the system – both in the presence of the gravity loads considered concurrent with the design seismic action. If α_1 is taken as a multiplicative factor on seismic action effects from the elastic analysis for the design seismic action, its value may be computed as the lowest value of the ratio $(M_{Rd}–M_V)/M_E$ over all member ends in the structure. M_{Rd} in this case is the design value of the moment capacity at the member end; M_E and M_V are the bending moments there from the elastic analysis for the design seismic action and for the concurrent gravity loads, respectively. The value of α_u may be found as the ratio of the base shear at development of a full plastic mechanism according to a nonlinear static ("pushover") analysis (with the gravity loads concurrent with the seismic action maintained constant in the course of the analysis, while lateral forces monotonically increase, according to Section 4.6.1), to the base shear due to the design seismic action (Fig. 1.12). For consistency with the calculation of α_1, the moment capacities at member ends in the pushover analysis should be the design values, M_{Rd}. If the mean values of moment capacities are used instead, as customary in pushover analysis, the same values should also be used for the calculation of α_1.

In most cases the designer will not consider worth doing iterations of pushover analyses and design based on elastic analysis, just for the sake of computing the ratio α_u/α_1 for the q-factor. For this reason, Eurocode 8 provides default values of this ratio. For buildings regular in plan, the default values are:

– $\alpha_u/\alpha_1 = 1.0$ for wall systems with just two uncoupled walls per horizontal direction;
– $\alpha_u/\alpha_1 = 1.1$ for:
  • one-storey systems and frame-equivalent dual (i.e., frame-wall) ones, and
  • wall systems with more than two uncoupled walls in the horizontal direction considered.

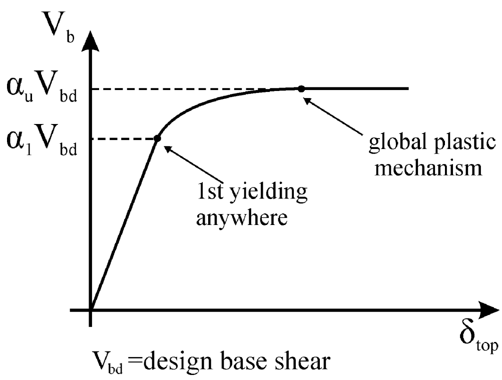

Fig. 1.12 Definition of factors α_u and α_1 on the basis of base shear v top displacement diagram from pushover analysis

– $\alpha_u/\alpha_1 = 1.2$ for:
  - one-bay multi-storey frame systems and frame-equivalent dual ones,
  - wall-equivalent dual systems, and
  - coupled wall systems.
– $\alpha_u/\alpha_1 = 1.3$ for multi-storey multi-bay frames or frame-equivalent dual systems.

In buildings which are irregular in plan according to the classification criteria of Eurocode 8 presented in Sections 2.1.5 and 2.1.6, the default value of α_u/α_1 is the average of:

– 1.0, and
– the default values given above for buildings regular in plan.

Values higher than the default ones may be used for α_u/α_1, up to a maximum of 1.5, provided that the higher value is confirmed through a pushover analysis, after design with the resulting q-factor.

The various types of structural systems that appear in the definition of the above default values of α_u/α_1 are defined in Eurocode 8 as follows:

In a "frame system" or in a "wall system" the seismic base shear taken, according to the analysis, by frames of beams and columns, or by walls, respectively, designed and detailed for earthquake resistance is at least 65% of the total. In-between "frame" and "wall" systems are the "dual systems". These are classified as "wall-equivalent dual" or as "frame-equivalent dual", if the fraction of the base shear resisted by walls is more, or less, than 50%, respectively. A wall system is considered as a "coupled wall system", if more than 50% of the total wall resistance is provided by coupled walls. According to Eurocode 8, two walls are considered as coupled, if they are connected together (normally at each floor level) through regularly spaced beams that meet special ductility conditions ("coupling beams") and this coupling reduces by at least 25% the sum of the bending moments at the base of the individual walls (the "piers"), compared to that of the two "piers" working separately.

For concrete buildings which are characterised as regular in elevation according to criteria 1–6 in Section 2.1.7, Eurocode 8 specifies the values of the q-factor given in Table 1.1.

Table 1.1 Basic value, q_o, of behaviour factor for heightwise regular concrete buildings in Eurocode 8

Lateral-load resisting structural system	DC M	DC H
Inverted pendulum system	1.5	2
Torsionally flexible structural system	2	3
Uncoupled wall system, not belonging in one of the two categories above	3	$4\alpha_u/\alpha_1$
Any structural system other than those above	$3\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$

"Inverted pendulum systems" are defined as those with at least 50% of their total mass in the upper third of the height, or with energy dissipation at the base of a single element. One-storey frame systems with all columns connected at the top (via beams) in both horizontal directions and maximum value of normalised axial load ν_d in the combination(s) of the design seismic action with the concurrent gravity loads not greater than 0.3 are excluded. "Inverted pendulum systems" are entitled very low q-factors (the q-factor for those of DC M does not exceed the value of 1.5 available thanks to overstrength alone, without design for ductility), because of concerns for potentially large P-Δ effects or overturning moments and reduced redundancy. However, inverted pendulum buildings seem unduly penalised, in view of the q-factors of 3.5 assigned by Eurocode 8 to bridges with concrete (single-)piers and more than 50% of the mass at the level of the deck. To alleviate this penalty, Eurocode 8 permits increasing the value of q_o for inverted pendulum systems that are shown capable of energy dissipation in their potential plastic hinges higher than normal for their chosen Ductility Class.

A system is defined in Eurocode 8 as "torsionally flexible", if at any floor the radius of gyration of the floor mass exceeds the torsional radius in one or both of the two main directions of the building in plan. As emphasised in Section 2.1.6, such systems are sensitive to torsional response about a vertical axis.

The values of q in Table 1.1 are called basic values, q_o, of the q-factor. They are the ones linked to ductility demands and member detailing (see Chapter 5). For the calculation of the seismic action effects from linear analysis, the value of q is reduced with respect to q_o in the following cases:

- In buildings which are irregular in elevation according to the classification criteria of Eurocode 8 presented in Section 2.1.7, the q-factor value is reduced by 20%.
- In wall, wall-equivalent dual or "torsionally flexible" systems, the value of q is the basic value q_o (reduced by 20% if there is irregularity in elevation), multiplied by a factor equal to $(1 + \alpha_o)/3$, but with values between 0.5 and 1, where α_o is the (mean) aspect ratio of the walls in the system (sum of wall heights, h_{wi}, divided by the sum of wall cross-sectional lengths, l_{wi}). This factor reflects the adverse effect of a low shear span ratio on the ductility of walls. It is less than 1 if α_o is less than 2, which corresponds to a mean shear span ratio of the walls in the system less than 1.33 (squat, typically non-ductile walls).

The above reductions of q notwithstanding, DC M and H buildings are entitled to a final q-factor value of 1.5, considered to be always available owing to overstrength alone.

A building that is not characterised as an "inverted pendulum" or a "torsionally flexible system" can have different q-factors in the two main horizontal directions, depending on the structural system and its vertical regularity classification in these two directions, but not due to ductility class, which is the same for the entire building.

1.4.3.2 US Standards

The force reduction or response modification factor R depends on the structural system and its ductility. The force reduction factors R specified by US standards (BSSC 2003, SEAOC 1999, ICBO 1997, ACI 2008) are considered to be composed of the following factors:

- One factor due to system ductility, equal to the ratio of the total lateral force for elastic response, to the actual lateral force resistance at full yielding of the system.
- Another factor due to overstrength, denoted by Ω_o, and equal to the ratio of the actual resistance at full yielding to the prescribed design forces. This factor is the counterpart of the product $1.5\alpha_u/\alpha_1$ representing overstrength of materials, elements and the structural system in Eurocode 8.

The NEHRP provisions (BSSC 2003) set $\Omega_o = 3$ in frames and $\Omega_o = 2.5$ in those dual systems where the frame provides at least 25% of the lateral force resistance and in systems that carry gravity loads through a space frame and lateral loads via concrete walls ("building frame systems"). Inverted pendulum systems have $\Omega_o = 2.0$. The overstrength factor Ω_o is also used to calculate the design shear force using an alternative to capacity-design, namely as Ω_o times the value from linear analysis. This amounts to calculating the seismic moments from the composite R factor and the seismic shears from the part of the R factor which is due to system ductility alone. In (SEAOC 1999) $\Omega_o = 2.8$, except in inverted pendulum systems, where $\Omega_o = 2.0$.

The composite R factor depends on the structural system. Values quoted below for concrete buildings are according to BSSC (2003), with the (SEAOC 1999) values given in parenthesis:

- The highest value of $R = 8$ (8.5) is for "Special Moment Frames".
- "Intermediate Moment Frames" have $R = 5$ (5.5).
- "Ordinary Moment Frames" have $R = 3$, due to overstrength alone ($R = \Omega_o$).
- Systems where gravity loads are taken by a 3D frame ("building frame") and the full lateral resistance is provided by concrete walls have $R = 6$ (5.5) if the walls are of "special" ductility, or $R = 5$ if they are of "ordinary".
- Systems where gravity loads are taken by walls ("bearing walls") and the full lateral resistance is provided by the same or other concrete walls have $R = 5$ (4.5) for walls of "special" ductility or $R = 4$, for "ordinary".
- Dual systems where "Special Moment Frames" provide at least 25% of the lateral force resistance (with the rest provided by walls) have $R = 8$ (8.5) if the walls are of "special" ductility (coupled walls included), or $R = 6$ if they are of "ordinary".
- Dual systems where "Intermediate Moment Frames" provide at least 25% of the lateral force resistance (the rest being provided by walls) have $R = 6.5$ (6.5) for walls of "special" ductility (coupled walls included) or $R = 5.5$ for "ordinary".
- Inverted pendulum systems have $R = 2.5$ (2.2) if their columns are of "special" ductility, or $R = 1.25$ if the columns are "ordinary".

Recent efforts to rationalize the R factor of US codes through system ductility and overstrength notwithstanding, the R values are still based on performance in past earthquakes and economic considerations. The R-factor values above were developed mostly on the basis of past performance of frames with multiple bays and with all their connections moment resisting. For reasons of economy and functionality, recent years have seen wider application of frames with fewer bays, supporting large floor areas. To counter the reduced redundancy of such systems, in buildings of "Seismic Design Category" D, E or F the R factor is reduced by a redundancy factor ρ, taking values between 1.0 and 1.3 (BSSC 2003) or 1.5 (SEAOC 1999). In (SEAOC 1999) ρ is the largest calculated in all storeys within the lower two-thirds of the building. Its value increases with increasing floor area and with the maximum (over all storeys for a given horizontal direction) fraction, $r_{\max}$, of a storey shear resisted by a single component (see Section 2.1.9 and Eqs. (2.3) and (2.4) for details). In dual systems with at least 25% of the lateral force resisted by the frame (SEAOC 1999) reduces the so-computed ρ-value by 20%. In (BSSC 2003) ρ is equal to 1.0, unless any storey where the storey shear exceeds 35% of the base shear depends on a single wall or pier of a coupled wall (including their connection to the rest of the lateral load resisting system) or on (both ends of) a single beam, for more than one-third of the storey's shear resistance or for the storey's torsional regularity (with a regular storey defined as one where the interstorey displacement at any point on the perimeter does not exceed by 40% or more the average in the storey). In these other cases ρ is taken equal to 1.3 (BSSC 2003).