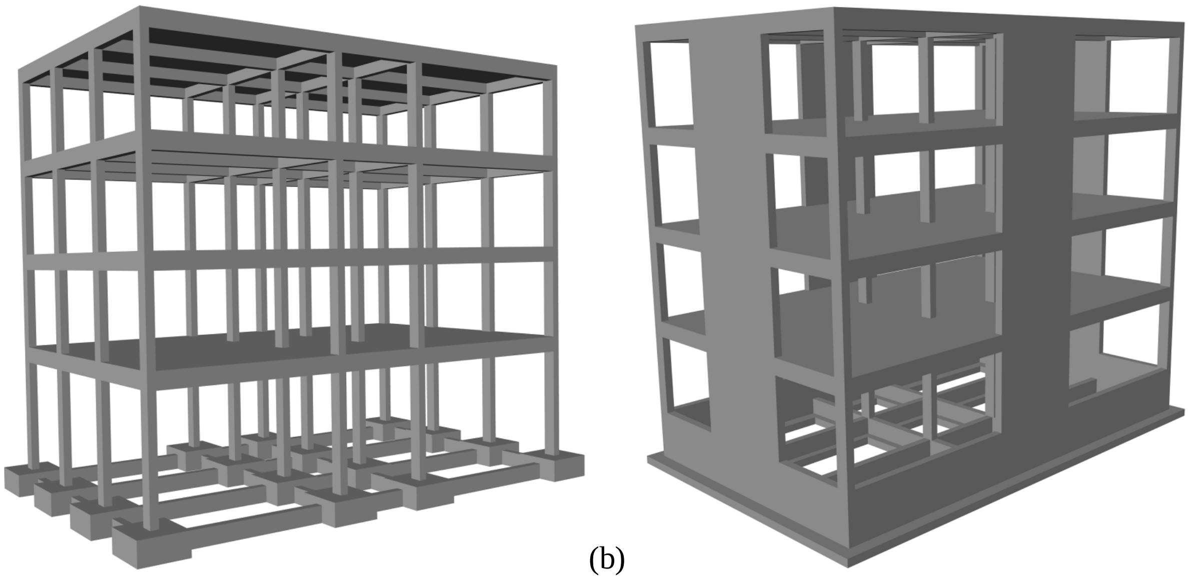
CHAPTER 4: CONCEPTUAL DESIGN OF CONCRETE BUILDINGS FOR EARTHQUAKE RESISTANCE

4.4 Structural systems of concrete buildings and their components

4.4.1 Introduction

The raison d'être of concrete buildings is to create horizontal surfaces for use/occupancy (floors) or protection (the roof). Most of the mass generating the inertia forces in an earthquake resides on these horizontal elements. Gravity loads are transferred from there to the ground via vertical elements, typically columns. Beams or girders span between columns, to facilitate the collection of gravity loads from the horizontal surfaces and their transfer to the columns (Fig. 4.11(a)). Concrete walls are often used to resist horizontal forces and to brace the building laterally against second-order (P-Δ) effects (Fig. 4.11(b), see Sect. 5.2.3.4 and Eq. (5.11) for the bracing role of walls under factored gravity loads).



(a) (b)

Fig. 4.11 Structural systems: (a) frame resisting both gravity loads and lateral actions, on footings with two-way tie-beams; (b) wall-frame lateral load resisting system on two-way foundation beams.

Concrete walls can resist a horizontal earthquake very efficiently, working as vertical cantilevers. However, unlike in masonry buildings, it is not cost-effective to collect from the floors and transfer to the ground all gravity loads through what is called in Eurocode 8 "ductile" concrete walls. Note, though, that it may be cost-effective to use both for seismic and gravity actions only "large, lightly reinforced walls" per Eurocode 8, highlighted in Section 4.4.2.1 but not covered in detail in this book.. Ductile walls normally complement a combination of columns and floor beams, whose main role is to support the gravity loads acting on the horizontal surfaces of the building (Fig. 4.11(b)). Normally the beams are directly and rigidly connected to the columns. The resulting moment-resisting beam-column frame is also efficient in resisting horizontal or vertical earthquake forces within its plane. So, besides their main role as a gravity-load resisting system, frames of beams and columns double as earthquake resisting systems; in fact, frames are the most common type of such a system in concrete buildings.

Inertia forces should find their way to the foundation via a smooth and continuous path in the structural system. From that point of view, cast-in-situ concrete is better for earthquake-resistant buildings than prefabricated elements – of concrete, steel or timber – assembled on site: the connections between such elements create discontinuities and potentially weak points in the flow of forces. So, cast-in-situ construction is the technique of choice for earthquake resistant concrete buildings, at least in high seismicity regions.

4.4.2 Ductile walls and wall systems

4.4.2.1 Concrete walls as vertical cantilevers

A wall differs from a column in that, under lateral loading, it works as a vertical cantilever. A column, by contrast, needs to be combined with beams into a frame, in order to resist lateral loads efficiently: its moment resistance at the base is too small to make a meaningful contribution to the base shear of the building, if divided by the shear span (moment-to-shear ratio) of a vertical cantilever. Moreover, its lateral stiffness as a vertical cantilever is too low to be effective in reducing interstorey drifts for damage limitation (see Section 1.3.2) or P-Δ (second-order) effects (see Section

3.1.12). To play its role as a vertical cantilever, the wall must be much stiffer than any beams it may be connected to at floor levels, so that these beams act only as parts of the horizontal diaphragm through which the wall receives the lateral forces from the floor, and not as horizontal elements of a frame encompassing both the wall and these beams. So, the wall's bending moment diagram under lateral loading looks like that of a vertical cantilever (see Fig. 5.6 and Figs. 7.17, 7.19, 7.24, 7.25 in the example building of Chapter 7): the moment does not change sign within a storey (except possibly near the top of wall-frame systems); the moments decrease considerably from the wall base to the top, much more than the shears do. Besides, if beams frame into the wall at floor levels, the wall bending moment is normally larger right above a floor than right below it; as the same vertical bars cross these two sections and the increase in axial compression enhances the wall moment resistance, plastic hinges can form in the wall only above floor levels. Multiple plastic hinging may well develop up the height of the wall, if the wall moment resistance at floor levels and at the connection to the foundation is tailored to the elastic seismic moment demands. Even then, a soft-storey mechanism cannot form in the wall itself, as it requires plastic hinging in counterflexure at two different locations up the height of the wall (cf. Fig. 2.9(a), (d), (e) and Fig. 2.12).

To ensure that a wall plays the role of a stiff and strong vertical spine of the building and prevents a soft-storey mechanism, Eurocode 8 promotes localisation of the wall inelastic deformations at its base. A wall designed and detailed to dissipate energy in a single flexural plastic hinge at the base and remain elastic throughout the rest of its height is called in Eurocode 8 “ductile wall”. It is the main wall type addressed in Eurocode 8, but not the only one. An alternative is allowed, termed “large lightly reinforced wall”, where flexural overstrength over the seismic demands from the analysis is intentionally avoided anywhere up the height of the wall, in order to promote plastic hinging at several floor levels above the base and translate the global displacement demand into small rotation demands at several locations up the wall. The inelastic deformation demand at the base of the wall is thus reduced; it may even be eliminated, by allowing rocking of the wall's footing, instead of fixing the base of the wall against rotation – a prerequisite for plastic hinging at the base of a "ductile wall". In this way, the cumbersome and expensive detailing of the wall base region for ductility is avoided.

Large lightly reinforced walls have certain advantages that ductile walls lack; for instance, rocking of a long footing and/or rotation of a long wall section about a neutral axis close to the compression edge of the wall raise the centroid of the wall section and, with it, the weights supported by the wall, cyclically (but temporarily) converting part of the vibration energy into recoverable and harmless potential energy of these weights, instead of inelastic deformation energy in plastic hinges, associated with permanent deformations and damage. Therefore, systems of large lightly reinforced walls designed according to their own special rules in Eurocode 8 may be more cost-effective under certain conditions than systems of ductile walls per Eurocode 8. However, as the use of large lightly reinforced walls is not common yet, this book covers only ductile walls.

4.4.2.2 What distinguishes a wall from a column?

Design codes define a wall as a vertical element with an elongated cross-section: a lower limit of 4 for the aspect ratio (long-to-short dimension) of a rectangular cross-section is used in Eurocode 2 for a vertical element to be considered as a wall. If the cross-section consists of rectangular parts, one of which has an aspect ratio greater than 4, the element is also classified as a wall. With this definition on the basis of the cross-sectional shape alone, a wall differs from a column in that it resists lateral forces mainly in one direction (parallel to the long side of the section) and can be designed for such a unidirectional resistance by assigning the flexural resistance to the two edges of the section ("flanges", or "tension and compression chords") and the shear resistance to the "web" between them, as in a beam. So, for the purposes of moment resistance and deformation capacity, the designer may concentrate the vertical reinforcement and provide concrete confinement only at the two edges of the section. Note that, if the cross-section is not elongated, the vertical element has to develop significant lateral force resistance in both horizontal directions; so, it is meaningless to distinguish the "flanges", where the vertical reinforcement is concentrated and the concrete confined, from the "web", where they are not.

The above definition of a "wall" is appropriate for dimensioning and detailing at the level of the cross-section, but meaningless for the intended role of a wall in the lateral load resisting system and for the usual practice to design, dimension and detail the wall as an entire element and not just at the cross-sectional level. Seismic design often relies on walls for the prevention of a storey-mechanism in the plane parallel to the wall's long direction, without checking if plastic hinges form in beams rather than in columns. However, walls can impose a beam-sway mechanism only if they act as vertical cantilevers (i.e. if their bending moment has the same sign throughout at least the lower storeys) and develop a plastic hinge only at the base. Whether a wall, as defined above, will indeed act as a vertical cantilever and form a plastic hinge only at its base does not depend on the aspect ratio of its section, but on how stiff and strong the wall is relative to the beams it is connected to at storey levels; if these beams are almost as stiff and strong as the wall, then the wall works as a frame column rather than as a vertical cantilever. For a wall to play its intended role, the length dimension of its cross-section, l_w, should be large, not just relative to its thickness, b_w, but in absolute terms. To this end, and for the beam sizes commonly found in buildings, a value of at least 1.5 m for low-rise buildings or 2 m for medium- or high-rise ones is recommended for l_w. In fact, it can be shown (Fardis 2009) that the optimal value of l_w for moment and shear resistance, stiffness and ductility is about one-sixth of the total height of the wall, H_{tot}.

4.4.2.3 Conceptual design of wall systems

The walls of a wall system should be arranged in two orthogonal horizontal directions with as much two-way symmetry as possible. If the individual walls are all similar and symmetrically placed, they will be subjected at every storey to fairly uniform seismic force and deformation demands, minimising the uncertainty about the seismic response. In a system with (very) dissimilar walls, the stronger and stiffer ones will yield first, imposing on the rest their inelastic deflection pattern, notably one where storey drifts increase almost linearly to the top owing to the rotation of the plastic hinge at the base, while the walls that are still elastic tend to deflect as vertical cantilevers. In that case, besides the increased uncertainty of the post-elastic response, the floor diaphragms will be stressed hard to iron out the differences in heightwise deflection patterns between the stiffer walls, which have gone inelastic, and the more flexible ones, which remain elastic. Note though that the price of complete uniformity is poor redundancy: plastic hinges will develop almost simultaneously at all wall bases and there will be little overstrength or redistribution of forces from certain walls to others.

Almost all our knowledge of the cyclic behaviour of concrete walls concerns walls with a two-way-symmetric rectangular or quasi-rectangular section (barbelled section, i.e., rectangular with each edge widened into a rectangular or square “column” or compact flange – with an aspect ratio less than 4 – to enhance the moment resistance and prevent lateral instability of the compression zone). Such walls are modelled and dimensioned as prismatic elements having an axis through the centroid of the section. Lacking a better alternative, the same practice is applied when a rectangular wall runs into or crosses another wall at right angles, to create a wall with a composite cross-section of more than one rectangular parts – each part with an aspect ratio greater than 4 (L-, T-, U-, H-shaped walls, etc). Such walls have high stiffness and strength in both horizontal directions, hence are subjected to biaxial bending and bi-directional shears during the earthquake. They are more cost-effective than the combination of their constituent parts as individual rectangular walls. However, present-day knowledge of their behaviour under cyclic biaxial bending and shear is very limited, and the rules used for their dimensioning and detailing still lack a sound basis. Moreover, their detailing for ductility is complex and difficult to implement on site. For this reason, it is recommended to make limited use of such walls in practical design. If non-rectangular walls are chosen, they should have a fairly simple section (e.g. one-way-symmetric U, or two-way-symmetric H).

Large openings should be avoided in ductile walls, especially near the base, where the plastic hinge forms. If they are necessary for functional reasons (doors or windows), they should not be staggered vertically, but should be arranged at every floor in a regular pattern, creating a coupled wall, with the lintels between the openings serving as coupling beams and designed as such. According to Eurocode 8, two walls are considered as coupled, if they are connected together (normally at each floor) through regularly spaced beams meeting 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 independently.

4.4.2.4 Advantages and disadvantages of walls for earthquake resistance

Structural systems dominated by ductile walls have many advantages for earthquake resistance:

- The high lateral stiffness of walls reduces interstorey drifts and structural or non-structural damage; it also overshadows the contribution of masonry infills to the lateral stiffness of the building and reduces the adverse effects: global ones, due to their potential irregularity in plan (eccentric placement) or elevation (open storey(s)), or local, notably shearing off weak columns, the creation of captive, squat columns, etc.
- Soft-storey mechanisms are precluded by the absence of wall counter-flexure within a storey.
- Rocking of the wall's footing or of the part of the wall above a plastic hinge raises the supported weights and is favourable for seismic performance.
- Overall, systems of walls are more cost-effective for earthquake-resistance than beam-column frames.

There are drawbacks as well:

- Walls are inherently less ductile than beams or columns, more sensitive to shear and harder to detail for ductility.

- The small number of walls required for earthquake resistance leads to smaller redundancy and fewer alternative load paths.
- It is difficult to place several long walls without compromising the architectural function of the building, producing large eccentricities in plan, or creating a torsionally sensitive building (i.e., one with more lateral stiffness closer to the centre in plan than to the perimeter).
- It is not cost-effective to support the building's gravity loads with walls alone; certain beams and columns are needed anyway for that and may efficiently serve earthquake resistance as well.
- It is hard to provide an effective foundation to a wall, especially with isolated footings. Because of the large bending moment and the relatively low vertical load of walls, the development of tensile forces in the foundation is often inevitable. A more favourable situation is to have the wall continue downwards from the ground floor into a basement (see Sections 4.4.5, 6.3.1 and 7.1). In such a case, the wall bending moment decreases within the basement from its maximum value at ground level (see Figs. 7.17, 7.24, 7.25), owing to the lateral restraint (horizontal forces) that the basement floors provide; hence the moment applied at the foundation level may be substantially smaller, and vertical tension forces are avoided. The downside is that the sharp decrease of the wall's bending moment below the ground floor entails development of large shear forces in the wall (see Figs. 7.17, 7.24, 7.25).
- There is some uncertainty concerning certain features of the seismic response of walls and their systems: the cyclic behaviour and seismic performance of individual walls (which is more difficult to study experimentally or analytically than in the case of beams or columns); the rocking response and the associated lifting of the weight supported by the wall, the increase of wall shears after plastic hinging at the base (see Section 5.6.2.1) etc. Moreover, modelling for analysis, and dimensioning/detailing of walls is more challenging compared to frame columns (especially for non-rectangular walls).

4.4.3 Moment-resisting frames of beams and columns

4.4.3.1 Special features of the seismic behaviour of frames - The role of beam-column connections

In a lateral-load-resisting system comprising only uncoupled walls, the sum of the wall seismic shears at the base is equal to the resultant of the lateral seismic forces applied at storey levels (: seismic "base shear" of the building); the resultant moment of these storey lateral seismic forces with respect to the base (: seismic "overturning moment" at the base) is equal to the sum of bending moments at the base of the walls. So, walls resist the seismic overturning moments and shears directly, through bending moments and shears, respectively, in the walls themselves. In contrast, frames resist the seismic overturning moment not by the column moments, but through their axial forces (tensile at the windward side of the plan, compressive at the opposite, leeward one, see Figs. 7.10, 7.13, 7.16, 7.20, 7.23). The column bending moments resist indirectly the seismic storey shears: the algebraic difference of bending moments at the top and bottom of each column produces its contribution to the seismic shear of the storey. So, the seismic response of frame members is governed by flexure, or strictly speaking by normal action effects: bending moments and axial forces.

Elastic moment, shear and axial force diagrams due to the seismic action in the frames of the example building of Chapter 7 are depicted in Figs. 7.8 to 7.16 and 7.20 to 7.23. Among other features, the seismic moment diagrams from the "lateral force method" exhibit an abrupt change in the algebraic value of the seismic moment across any beam-column connection: the beam or column moment turns from large and positive at one face of a joint into large but negative at the opposite face. If the joint, being of finite dimensions, is considered as the part of the beam within the column, this abrupt change in the beam moment across the joint means that a large vertical shear force develops inside it, which is equal to the sum of the absolute values of beam moments at the joint faces divided by the column width in the plane of the frame (Fig. 4.12(a)). By the same token, if the joint is considered as the part of the column between adjacent beam spans, the abrupt change in column moments across the joint implies a large horizontal shear force in it, which is equal to the sum of absolute values of column moments at the joint faces divided by the beam depth (Fig. 4.12(b)).

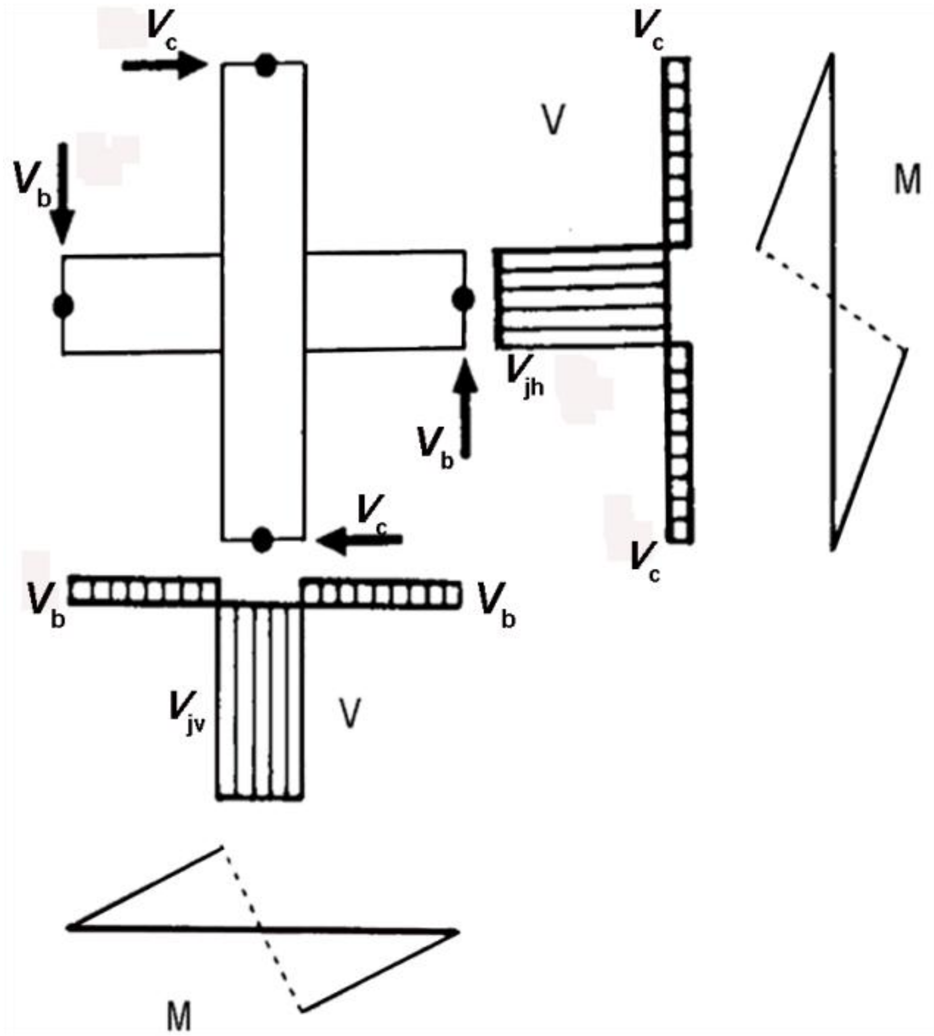


Fig. 4.12 Seismic moments and shears in the beams and columns connected at a joint and seismic shears in the joint core: (a) joint considered as part of the beams; (b) joint considered as part of the column.

So, the core of the joint is subjected to very high shear stresses, equal to the sum of (the absolute values of) the beam or column seismic moments at opposite faces of the joint divided by the volume of this core (see Fig. 2.21). Another repercussion of the rapid change in the algebraic value of seismic moments across any beam-column connection is that any beam or column longitudinal bars crossing the joint are under high tensile stresses on one side of the joint and under high compressive stresses on the other. This means that very high bond stresses develop all along the stretch of such bars within the joint; if plastic hinges form in the beam or the column at both sides of the joint, the value of these bond stresses may exceed the bar yield force divided by the lateral surface of the bar inside the joint (as a matter of fact, along the bottom bars, bond stresses may approach twice that value). Because the beams, rather than the columns, are expected to develop plastic hinges, to accommodate these bond stresses the column width should exceed a certain multiple of the beam bar diameter, as specified in Section 5.2.3.3 of Chapter 5. This often turns out to be a major constraint on the column size or the beam bar diameter. If the relevant rule in Eurocode 8 is not met, the bars may slip through the joint, thus increasing the apparent flexibility of the members framing into it and preventing them from plastic hinging next to it (Fig. 2.22(a)). Although this will not have catastrophic consequences, it prevents the frame members connected to that joint from contributing to the strength, stiffness and energy dissipation capacity of the frame to their full potential.

4.4.3.2 Conceptual design of RC frames for earthquake resistance

The general layout and certain details of the geometry of an individual plane frame have a major impact on its seismic behaviour. Very important also is the overall layout of the frames in a frame structural system. The location of frames in plan and their span lengths are normally governed by architectural and functional considerations, while beam depths may be controlled by design for factored gravity loads (for the "persistent and transient design situation" of EN 1990). Nevertheless, the structural designer is essentially free to choose the all-important geometric details of individual plane frames and has certain freedom concerning their overall geometry and location in plan.

Any single plane frame should run continuously from one side of the building plan to the other, without offsets, interruptions (i.e. missing beams between adjacent columns in a floor), or indirect supports of beams on other beams:

- If a beam does not continue straight from span to span, but its axis is offset at the column between them, there is no smooth flow of beam internal forces through a proper beam-column joint, neither continuity of the beam longitudinal bars across the column from one span to the next: these bars have to terminate there and be separately anchored at the joint.

- Even when the beam axis is not offset from span to span, the smooth flow of internal forces from the beam(s) to the column is impaired by a large eccentricity between the axis of the beam and the supporting column. The behaviour of strongly eccentric beam-column joints is, by-and-large, unknown. For that reason, Eurocode 8 sets an upper limit on the eccentricity, e, between the axis of the beam and the column at their connection:

$$e \leq b_c/4 \tag{4.10}$$

where b_c is the largest cross-sectional dimension of the column at right angles to the beam axis. Note that, if one lateral side of the beam is flash with one face of the column, this condition restricts the ratio of b_c to the beam width, b_w, not to be greater than 2.0. This is the case at the corner columns of the example building in Chapter 7 (see Fig. 7.2).

- If a beam terminates at an (indirect) support on another beam, there is large uncertainty concerning its rotational restraint by the supporting beam via torsion in the latter. In approximation, an indirect support may be considered as a simple support; the indirectly supported beam is less effective in frame action than one connected to columns at both ends.

The ideal plane frame has:

1. constant beam depth in all bays of a storey,
2. constant size of each column in all storeys,
3. (about) uniform spans,
4. interior columns of (about) the same size,
5. (about) the same height in all storeys.

Note that, if points 1 to 4 above are met and the exterior columns have one-half the stiffness of the interior ones, then, if the effect of column axial deformations is negligible, all beams in the storey will develop the same elastic seismic shears and bending moments (which will be equal at the two beam ends); all interior columns will also have the same elastic seismic shears and moments while their elastic seismic axial forces will be zero; the two exterior columns will develop half the seismic elastic shears and moments of interior ones and will resist the full seismic overturning moment, via seismic axial forces equal to the seismic overturning moment at storey mid-height divided by the distance between the axes of the two exterior columns. If all members of such a frame are dimensioned to resist exactly the elastic seismic moments, all beam ends in a storey will be subjected to (about) the same inelastic chord rotation demands; all columns, interior or exterior, will also develop (about) the same inelastic rotation demands at storey bottoms; the same at column tops. Such uniformity reduces uncertainty concerning the distribution of seismic action effects among frame members. If the two exterior columns have more than one-half the stiffness of interior ones, their share of storey elastic seismic shears will increase (alongside their elastic moments, as well as those at the two outer beam ends), but less than proportionally; seismic axial forces in interior columns will be non-zero, but small.

Beams with long span may have their top reinforcement at the supports governed by factored gravity loads (the "persistent and transient design situation" per EN 1990), rather than by the "seismic design situation". This will result in beam overstrength, $M_{\mathrm{Rd,b}}$, relative to the moment demand, $M_{\mathrm{Ed,b}}$, in the "seismic design situation". The overstrength is carried over to the capacity design of columns around joints (per Eq. (5.31) in Section 5.4.1 of Chapter 5) and the capacity design shears of beams and columns (per Eqs. (5.42) and (5.44), respectively, in Section 5.5.1), penalising them and creating some uncertainty whether plastic hinges will form in the beams or the columns. Besides, the large hogging moments due to quasi-permanent gravity loads at the ends of long span beams may prevent reversal of yielding in sagging flexure at any plastic hinges that may form there. As a result, inelastic elongations accumulate in the top reinforcement and the beam gradually grows longer, pushing out the supporting columns and possibly forcing exterior ones to separate from exterior beams at right angles to the elongating one(s).

Beams with low span-to-depth ratio have to be dimensioned for high shear forces, from the seismic analysis or from capacity design in shear (see Eqs. (5.42) in Section 5.5.1). At the ends of such beams the shear due to quasi-permanent gravity loads is small (see last term in Eqs. (5.42)) and reversal of the seismic action will also cause an almost full reversal of the sign of the acting shear (cf. Eq. (5.43) in Section 5.5.1), exhausting the shear capacity of the beam in both diagonal directions or causing sliding shear failure along through-depth cracks at the end section(s) of the beam. To resist such effects, diagonal reinforcement or stirrups at $\pm 45^{o}$ are needed at the ends of short beams (see Eqs. (5.42), (5.43) in Section 5.5.1 and (5.49)-(5.51) in Section 5.5.3). Moreover, unless diagonally reinforced, short beams have low deformation capacity and poor ductility.

For the reasons detailed above, short beam spans should be avoided, while spans of 4 to 5 m should be preferred over longer ones, at least for the storey heights and gravity loads commonly encountered in buildings.

In frame systems (with the frames preferably having an individual geometry according to the above), frames should be arranged in two orthogonal horizontal directions in a way that maximises two-way symmetry and minimises irregularities in plan of the type highlighted in Section 4.3.2. If such frames are all similar and symmetrically placed, they will be subjected at every storey to fairly uniform seismic force and deformation demands, without undue concentration in a single frame, member or location thereof and risk of early failure. Whatever has been said in the first paragraph of Section 4.4.2.3 concerning dissimilar walls in a wall system applies by analogy to systems of frames with very different strength and stiffness: the stronger and stiffer ones will yield earlier during the response, imposing on the rest a deflection pattern where storey drifts increase almost linearly to the top, instead of following the storey shear force pattern. The floor will be subjected to larger in-plane forces, to bridge the differences between the drift patterns of the stiffer and already inelastic frames and those of the more flexible ones, which remain elastic. Complete uniformity will again result, though, in a less progressive formation of the overall plastic mechanism, and plastic hinges will develop almost simultaneously in the various frames, be it where expected. Moreover, the storeys have little overstrength after the first plastic hinge formation and cannot redistribute forces from certain locations to others.

4.4.3.3 Advantages and drawbacks of frames for earthquake resistance

The advantages of RC frames for earthquake resistance may be summarised as follows:

- Frames place few constraints on a building's architectural design, including the façade.
- Frames may be cost-effective for earthquake resistance, because beams and columns are placed anyway for gravity loads; so, they may also provide earthquake resistance in both horizontal directions, if their columns are large.
- Two-way frame systems, comprising several multi-bay plane frames per horizontal direction, are highly redundant, offering multiple load paths.
- Thanks to their geometry (notably their slenderness), beams and columns are inherently ductile, less prone to (brittle) shear failure than walls.
- Frames with concentric connections and regular geometry have well known and understood seismic performance, thanks to the numerous experimental and analytical studies carried out in the past; moreover, they are rather easy to model and analyse for design purposes.
- It is easier to design an earthquake-resistant foundation element for a smaller vertical member than for a larger one (i.e., for a column in comparison to a wall).

There are disadvantages as well:

- Frames are inherently flexible; the cross-section of their members may be governed by the interstorey drift limitation under the moderate earthquake for which limitation of damage to structural and non-structural elements is desired (see Section 1.3.2).
- Column counter-flexure in the same storey allows soft-storey mechanisms (Fig. 2.9(a)) which

lead to collapse.

- Earthquake resistance requirements on frames lead to large columns.
- The reinforcement detailing of frames for ductility requires workmanship of high level for its execution and good supervision on site (especially to fix the dense reinforcement and place/compact the concrete through the beam-column joints in two-way frames).
- Sizing and detailing of beam-column joints for bond and anchorage of beam bars crossing them is quite challenging. Difficulties increase with the use of higher strength materials, as the size of joints made of higher concrete strength is smaller, while higher steel strength implies higher bond stresses.
- There is still some uncertainty concerning the seismic response and performance of frames:
  - the effects of eccentric connections or strongly irregular layouts in 3D;
  - the size of the effective slab width in tension (see Fig. 2.22(b)) and the extent to which slab bars in it and parallel to the beam increase its flexural capacity for hogging moment, $M_{Rd,b}^{-}$, hence the beam capacity design shears per Eqs. (5.42) in Section 5.5.1 of Chapter 5 and the likelihood of plastic hinging in the columns, despite meeting the capacity design rule around joints per Eq. (5.31) in Section 5.4.1;
  - the behaviour of columns of two-way frames under cyclic biaxial bending with varying axial force, which may even cause plastic hinging in columns which meet the capacity design rule of Eq. (5.31) in separate uniaxial bending per horizontal direction.

4.4.4 Dual systems of frames and walls

4.4.4.1 Behaviour and classification per Eurocode 8

Walls and frame systems each have their advantages and disadvantages as lateral-load-resisting systems. Walls seem to have a better balance of advantages against drawbacks; nevertheless, a concrete building always has beams and columns to support the gravity loads; it is a waste not to use them for its earthquake resistance. So, frames and walls may well be cost-effectively combined in a single lateral-load resisting system.

Eurocode 8 uses the fraction of the elastic seismic base shear taken by all the system's frames according to linear analysis for the seismic action, to distinguish whether frames or walls dominate the lateral-load resisting system:

- When frames or walls take at least 65% of the seismic base shear, we have a “frame system” or a “wall system”, respectively.
- When the percentage of the seismic base shear taken by frames or walls is between 35 and 65%, the frame-wall system is called “dual”; if the fraction of elastic base shear taken by the walls is from 50 to 65%, the system is a “wall-equivalent dual”; if it is between 35 and 50%, it is a “frame-equivalent” one.

Eurocode 8 considers a wall system as a “coupled wall system”, if coupled walls, as defined at the end of Section 4.4.2.3, provide more than 50% of the total wall resistance.

The building in Chapter 7 is classified as a “wall-equivalent dual” in the X-direction and as a "wall system" one in Y (see Section 7.3.1).

Dual systems combine the strength, stiffness and immunity to soft-storey effects of wall systems with the ductility, deformation capacity and redundancy of frames. The walls prevent nonstructural damage in frequent, moderate earthquakes, helping the building meet the interstorey drift limits of Eurocode 8 under the damage limitation earthquake (Section 1.3.2). The frames serve as a second line of defence in strong earthquakes, in case the deformation capacity of the less ductile walls is exhausted and some walls lose part of their strength and stiffness.

The way frames and walls share the horizontal seismic action comes out of their different horizontal deflection pattern under lateral loading:

- Frames have a shear-beam-type of lateral displacement pattern, in which interstorey drifts follow the heightwise pattern of the storey seismic shears: they decrease from the base to the top.

- Walls fixed at the base deflect like vertical cantilevers: their interstorey drifts increase from the base to the roof.

If frames and walls are combined in the same structural system, the floor diaphragms impose on them common floor displacements. As a result, the walls restrain the frames at lower floors, taking the full inertia loads of these floors, while near the top the frame is called upon to resist the full floor inertia loads and, in addition, to hold back the walls, which – if alone – would have developed a large deflection at the top. So, in rough approximation, the walls of dual systems may be considered to be subjected to:

- the full inertia loads of all floors, and
- a concentrated force at roof level, in the reverse direction with respect to the peak seismic response and the floor inertia loads.

The concentrated force at the top exceeds the resultant inertia loads in the upper floors, i.e., the storey seismic shear there. So, the walls are often in reverse bending and shear in the upper storeys with respect to the storeys below (see Figs. 7.17, 7.24 and 7.25). If the frame is considered to be subjected to just the concentrated force at the top, equal and opposite to the one it applies there to the wall(s) and in the same sense as the floor inertia loads, then it has in all storeys a roughly constant seismic shear and about the same bending moments (see Figs. 7.8, 7.9, 7.11, 7.12, 7.14, 7.15, 7.21, 7.22). Thus, even when the cross-sectional dimensions of frame members are kept the same in all storeys, their reinforcement requirements for the seismic action do not decrease from the base to the top. As a matter of fact, the reinforcement required in the columns may even decrease in the lower storeys, thanks to the favourable effect of the higher axial load on flexural strength. Therefore, in dual systems column size should never decrease in the upper storeys.

4.4.4.2 Conceptual design of dual systems

Dual systems have a more complicated seismic response than pure frame or wall ones. The resulting larger uncertainty concerning their seismic behaviour and performance may be their only drawback as a system. They are a quintessential example of systems of dissimilar subsystems; hence, whatever has been said in the first paragraph of Section 4.4.2.3 and the last one of 4.4.3.2 applies to them as well. Their conceptual design should aim to reduce the uncertainties arising from this feature. For instance, floor diaphragms should be thicker and stronger within their plane than what is required in pure frame systems. Another uncertainty arises from any rocking of the walls at the base, which will shift part of the storey shear from the walls to the frames. Rocking of wall footings with uplift is an intrinsically complex phenomenon, not reliably modelled in the context of seismic design practice. Its underestimation will lead to unsafe design of the frames, while its overestimation is unsafe for the walls.

Note that in a system consisting only of walls, the distribution of seismic shear between them will be practically unaffected by the rotation of the walls at the foundation level: rotations will mainly increase the absolute magnitude of storey drifts. The effect of footing rotation is even smaller in pure frame systems, practically affecting the seismic action effects only in the ground storey; moreover, such rotation is much smaller than in wall footings, because the higher axial load of the column resists uplift; more importantly, the smaller the cross-section of a vertical element compared to the plan dimensions of its footing, the smaller its rotation. So, it is dual systems that suffer from the increased uncertainty due to the rotations of footings with respect to the ground.

A prudent design of a dual system would reduce differential rocking. Ideally, this could be achieved by providing full fixity of walls and columns at the foundation level. However, full fixity is unfeasible, except at the top of a rigid basement (as in the example building of Chapter 7). In all other cases, great attention should be paid in the analysis phase to the modelling of soil compliance under the foundation elements, especially those of walls. Moreover, sensitivity studies should be carried out concerning the assumptions made and the values of properties used in the analysis.

Tall buildings often have a strong wall near the centre in plan, e.g., around a service core housing elevators, stairways, vertical piping, etc., and stiff and strong perimeter frames. In such systems outrigger beams may be used to advantage, increasing the global lateral stiffness and strength and mobilising the perimeter frames against the seismic overturning moment.

4.5 The capacity design concept

4.5.1 The rationale

The fundamental period of concrete buildings, *T*, is normally in the constant spectral pseudovelocity part of the response spectrum or beyond that part: $T \geq T_C$. As pointed out in Sections 3.2.2 and 3.2.3, in that range inelastic seismic displacements are roughly equal to the elastic ones ("equal displacement rule"). A prime target of seismic design is to apportion the given total seismic displacement demand to the various elements of the building, entrusting inelastic deformations only to those elements which can reliably withstand them, while keeping in the elastic range those which cannot. The tool for such a control of the inelastic seismic response is “capacity design”. This tool establishes a strength hierarchy among the individual elements which ensures that, along the full load path of the inertia forces to the foundation ground, the strength of the structural system is governed by ductile elements, not brittle ones. Although capacity design is implemented during the detailed design phase, its effectiveness depends strongly on the structural layout and member sizes chosen during conceptual design.

The elements to which the global displacement demands are channelled via capacity design are chosen using the following criteria:

1. The inherent “ductility” of the element: its capacity to sustain large inelastic deformations and dissipate energy in cyclic loading, without material loss of force-resistance.
2. Importance for the stability of other elements and the integrity of the whole: vertical elements are more important than horizontal; the foundation is the most important part of the system.
3. Accessibility and difficulty to inspect and repair.

On the basis of these criteria, a hierarchy of elements is established, which determines if and in which order they may enter the inelastic range during the seismic response. “Capacity design” is the tool to enforce this hierarchy. It works as follows:

The elements higher in the hierarchy are identified; their required design resistance is determined then not from the analysis, but via “capacity design”, i.e., using only equilibrium and the force capacities of those elements which are ranked as less important, more accessible, or inherently more “ductile” (hence the term “capacity design”), so that these latter elements exhaust their force resistance (yield) before the former do and indeed shield them from yielding.

4.5.2 The role of a stiff and strong vertical spine in the building

A prime aim of “capacity design” is to prevent a "storey-sway" plastic mechanism, in which inelastic deformations concentrate in a single storey (Fig. 2.9(a)) and may lead to failure and collapse of its vertical elements, triggering overall collapse. As, for given fundamental period *T*, the global inelastic displacement demand at roof level is roughly given ("equal displacement rule"), it should be uniformly spread to all storeys, instead of a single one. For this to be kinematically possible, the beam-column nodes along any vertical element should stay on the same line during the seismic response. To this end, vertical elements should (see Fig. 2.9(b) to (e)):

- stay in the elastic range up their full height, and
- rotate about their base, either at a flexural "plastic hinge" they form just above their connection to the foundation (Fig. 2.9 (b) and (d)), or by rigid-body rotation of their individual footings relative to the ground (Fig. 2.9 (c) and (e)).

Such a side-sway plastic mechanism is kinematically possible, only if plastic hinges also form at both ends of every single beam of the system (“beam-sway” mechanisms). This produces the widest possible spreading of the global displacement demand through the structural system and minimises the local deformation demands on individual members or locations.

If the intended distributed plastic hinging in Fig. 2.9 (b) to (e) takes place simultaneously throughout the structure, beam ends and the bases of vertical elements will develop a chord rotation (: angle between the normal to the member section at a member end and the chord connecting the two member ends) 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}). Besides, the chord rotation ductility factor demand at member ends (: peak chord rotation demand during the response, divided by the chord rotation at yielding of that end) is roughly equal to the demand value of the top displacement ductility factor, μ_δ. Under the design seismic action, μ_δ is about equal to q_μ (see Eq. (3.120)), i.e., well within the capacity of concrete members with end regions detailed for ductility per Section 5.7. So, in the context of protecting life and fulfilling the no-collapse requirement, the "beam-sway" mechanisms of Figs. 2.9 (b)-(e) make possible, relatively easily and economically, fairly high q-factor values.

In the "storey-sway" mechanism of Fig. 2.9 (a), all inelastic deformations take place in the single "soft-storey", with plastic hinging at both ends of all vertical elements in the storey in counterflexure. The chord rotation demands at the ends of these vertical elements approach the ratio of the roof displacement, δ, to the soft-storey height, h_i. So, they are H_{tot}/h_i times larger than those of a "beam-sway" mechanism. The chord rotation ductility factor is about equal to H_{tot}/h_i times the global displacement ductility factor, μ_δ, derived from the q_μ-factor via Eqs. (3.119), (3.120) (cf. Eqs. (3.122), (3.123) and Fig. 3.15 in Section 3.26). No mid- or high-rise building can withstand such demands in its columns.

To spread the global inelastic deformation demands to the entire structural system and prevent a "soft-storey", the building needs a strong and stiff spine of vertical elements, which by virtue of their geometry and/or design will stay elastic above their base under any earthquake. This is achieved by overdesigning them (except at the base section) relative to the horizontal ones and/or the action effects from the analysis. Sections 5.4.1 and 5.6.1.1 present in detail how this is pursued through "capacity design" of columns or walls, respectively.

In addition to their vital role in spreading the total deformation and energy dissipation demands to the entire structural system, vertical elements also meet prioritisation criteria 1 and 2 of Section 4.5.1 for choosing which elements to capacity-design; compared to beams, they are:

- inherently less “ductile”, because axial compression adversely affects ductility; and
- more important for the stability and integrity of the whole structure.

However, concerning criterion 3, columns are easier to repair than beams, as they are accessible from all sides.

Eurocode 8 promotes beam-sway mechanisms through multiple means, direct or indirect:

- In frame- or frame-equivalent dual systems: by capacity design of the columns to be stronger in flexure than the beams and, therefore, escape plastic hinging (see Section 5.4.1).
- In wall- and wall-equivalent dual systems: by overdesigning them above the base, to remain elastic in flexure (see Section 5.6.1.1) and by entitling them to *q*-factor values comparable to those of frame- or frame-equivalent dual systems (see Section 4.6), despite their poorer redundancy and the inherently lower ductility of walls.
- Through the Eurocode 8 limits on interstorey drifts (computed for elastic response to the damage limitation seismic action, using the cracked stiffness of concrete members, see Section 1.3.2): these limits cannot be met without walls or good-size columns.

4.5.3 Capacity design in the context of detailed design for earthquake resistance

Capacity design is applied as follows in the context of detailed seismic design using linear analysis with the *q*-factor (cf. Section 5.1.1):

1. Detailed design starts with dimensioning for the Ultimate Limit State (ULS) in flexure (for the bending moment and axial force pairs from the analysis for all applicable ULS design situation) and detailing of the longitudinal reinforcement at those locations which are considered as appropriate/convenient to detail for cyclic ductility and energy dissipation and where flexural

plastic hinges are foreseen/allowed in a "beam-sway" plastic mechanism (called "dissipative zones" in Eurocode 8):

a) all beam ends connected to vertical elements (see Sections 5.3.1, 5.3.2, 5.7.1, 5.7.3, 5.7.4);

b) the base section of all vertical elements (at the connection to the foundation, see Sections 5.4.2 for columns, 5.6.1 for walls); and

c) the top and bottom regions of all columns which Eurocode 8 exempts from capacity design (see Sections 5.4.1 and 5.4.2).

2. All elements in shear and the regions of vertical elements outside "dissipative zones" in flexure are dimensioned to stay elastic after flexural yielding of the "dissipative zones". To this end, they are overdesigned with respect to the relevant action effects from the analysis, normally through "capacity design", employing equilibrium and the overstrength flexural capacities, $\gamma_{Rd}M_{Rd}$, of the already dimensioned "dissipative zones".
3. "Dissipative zones" are detailed to provide ductility capacity according to the deformation demands imposed on them by force-based design with the chosen q-factor.
4. The ground, and normally the foundation elements themselves, are normally capacity-designed to stay elastic when the "dissipative zones" in the superstructure reach their overstrength flexural capacities (Section 6.3.2); Eurocode 8 allows also the option to dimension and detail foundation elements for ductility, as in the superstructure, despite the difficulty of repairing them.

4.6 Ductility Classification

4.6.1 Ductility as an alternative to strength

According to Eqs. (3.119), (3.120) the design seismic forces are about inversely proportional to the global displacement ductility factor, μ_δ. So, increasing the available global ductility, reduces the internal forces for the dimensioning of structural members, hence possibly their cost. Apart from any cost benefits, ductility has several advantages as a substitute for strength:

- A high *q*-factor makes it feasible, or easier, to verify the foundation soil, which is normally made on the basis of strength, not of deformation capacity.
- Reduced strength serves as a physical upper limit on the inertia forces and the response accelerations that can develop in the structure, "isolating" from them, hence protecting, contents of the building and non-structural parts which are sensitive to acceleration.
- An ample ductility supply enhances robustness and resilience of the building to earthquakes stronger than the design seismic action and its sensitivity to the uncertain details of the ground motion.

However, high lateral force resistance, in lieu of enhanced ductility, offers other advantages:

- By helping the structure to stay elastic under more frequent, moderate earthquakes, higher strength reduces structural damage and improves usability after the event. Structural damage is also reduced under the design seismic action.
- Detailing of members just for strength, instead of ductility, is easier and simpler. It can be done more reliably, especially when the technical level of workmanship is not very high.
- Force-based design against non-seismic actions (including wind) provides certain lateral strength for free, to be used for earthquake resistance as well, without costly and demanding detailing of members for ductility.
- If the structural layout is unusually complex and irregular, outside the scope of seismic design standards addressing mainly ordinary layouts, the designer may feel more confident for his/her design by narrowing the gap between the results of linear analysis used to dimension the members and the nonlinear seismic response to the design seismic action, through a lower *q* value.

In view of the different advantages of both possible design choices (ductility vs strength), it is up to the designer to decide what is the best option for each specific situation at hand. In this context, and as explained in detail in the next section, Eurocode 8 introduces three different "Ductility Classes", leaving the choice to the designer. However, National Authorities may set some limitations to such a choice.

4.6.2 Ductility Classes in Eurocode 8

Eurocode 8 allows trading ductility for strength by providing rules for three alternative Ductility Classes (DCs):

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

4.6.2.1 Ductility Class L (Low): use and behaviour factor

Buildings of DC L are not designed for ductility; only for strength. Except certain minimum conditions for the ductility of reinforcing steel (see Table 5.6 in Section 5.7.2), they have to follow just the dimensioning and detailing rules specified in Eurocode 2 for non-seismic actions, e.g. wind. Although they are expected to stay elastic under the combination of the design seismic action and the concurrent gravity loads (the "seismic design situation"), they can use a behaviour factor value of q = 1.5 instead of q = 1.0, thanks to member overstrength due to (cf. Section 3.2.4):

– the difference between the mean strength of steel and in-situ concrete and the design values (: 5%-fractile strengths divided by partial material factors, see Section 5.1.2);
– the possible control of the amount of reinforcement in some critical sections by the requirements for non-seismic actions or by minimum reinforcement;
– the use of the same reinforcement at the cross-sections of a beam or column across a joint, determined by the most demanding of these two sections; rounding-up of the number and/or diameter of bars, etc.

DC L buildings are not cost-effective for moderate or high seismicity. Moreover, lacking engineered ductility, they may also lack a reliable safety margin against earthquakes stronger than the design seismic action. So, they are not considered suitable for moderate or high seismicity regions. Eurocode 8 recommends using DC L only for "low seismicity cases", but leaves this decision to the National Annex, along with the definition of what is a "low seismicity case": its recommendation is to consider it as such if the design ground acceleration on rock, a_g (including the importance factor, γ_I), does not exceed 0.08g, or the design acceleration on the ground, a_gS, is not more than 0.1g (see Section 3.1.3 for a_g and S).

Eurocode 8 allows also to use DC L if the seismic design base shear at the level of the foundation or the top of a rigid basement for q=1.5 is less than the base shear due to the design wind, or any other lateral action for which design is based on linear analysis.

4.6.2.2 Ductility Classes M (Medium) and H (High) and their use

Seismic design for lateral strength alone without engineered ductility is an extreme, for use only in the special cases highlighted in the Section above. In the prime case of seismic design, i.e., based on ductility and energy dissipation, Eurocode 8 gives the option to design for more strength and less ductility or vice-versa, by choosing between Ductility Class M or H.

Buildings of DC M or H have q-factor values higher than the default value of 1.5 used for DC L and considered as due to overstrength alone. DC H buildings enjoy higher values of q than DC M ones; in return, they are subject to stricter detailing rules (see Tables 5.1 to 5.5) and have higher safety margins in capacity design against shear (see Sections 5.5 and 5.6). However, unlike DC L, DC M does not systematically require more steel than DC H: the total quantities of materials are essentially the same; in DC H, transverse reinforcement and vertical members have a larger share of the total quantity of steel than in DC M.

DC M and H are expected to achieve about the same performance under the design seismic action, but DC M is slightly easier to design and implement and may give better performance in moderate earthquakes. DC H may provide larger safety margins than M against collapse under earthquakes (much) stronger than the design seismic action and may be more economic for high seismicity, especially if there is a strong local tradition and expertise in seismic design and on-site implementation of complex detailing.

Eurocode 8 does not relate the choice between DC M and H to seismicity or the importance of the structure, nor puts limits to their application. Countries are free to choose for the various parts of their territory and types of construction. They would better, though, leave this choice to the designer, depending on the specifics of the project.

4.6.3 Behaviour factor of DC M and H buildings

In Eurocode 8 the value of the behaviour factor, q, of DC M and H buildings depends on:

- the Ductility Class;
- the type of lateral-force-resisting-system; and
- the regularity or lack thereof of the structural system in elevation.

The value of the q-factor is linked, indirectly (through the ductility classification) or directly (see Section 5.7.3), to the local ductility and detailing requirements for members.

Table 4.1 lists the values of the q-factor for buildings which are regular in elevation per the Eurocode 8 criteria in Section 4.3. These values are called basic values, q_o, of the q-factor and are the ones linked to local ductility demands and member detailing (see Section 5.7.3). The value of q used for the calculation of the seismic action effects from linear analysis is reduced with respect to q_o:

1. In buildings irregular in elevation per Eurocode 8 (see Sections 4.3.4, 4.3.5): to $q = 0.8q_o$.
2. In wall, wall-equivalent dual or "torsionally flexible" systems, to $(1+\alpha_o)q/3 \geq 0.5q$, where q may be reduced per 1 above if there is irregularity in elevation, and α_o (≤ 2) 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 last reduction reflects the adverse effect of low shear span ratio on wall ductility for $\alpha_o < 2$ (a value corresponding to a mean shear span ratio of the walls in the system less than about 1.65, which are non-ductile).

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 thanks to overstrength alone.

Table 4.1 Basic value, q_o, of behaviour factor per EC8 for heightwise regular buildings

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

An “inverted pendulum system” is, per Eurocode 8, a building with at least 50% of the mass in the top third of its height, or with energy dissipation possible only at the base of one element (Eurocode 8 excludes from this category one-storey frame systems having all columns connected at the top through beams in both horizontal directions and a maximum value of normalised axial load, ν_d, among all combinations of the design seismic action with the concurrent gravity loads, which is less or equal to 0.3). The low q-factors of “inverted pendulum system” in row 1 are due to poor redundancy and sensitivity to P-Δ effects or overturning moments.

According to Eurocode 8, a system is "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 in plan. As pointed out in Section 4.3.2, it is also considered in Eurocode 8 as planwise irregular. Its low q-factor value in row 2 of Table 4.1 reflects the increased likelihood of twisting about the vertical, to which the perimeter elements of the building are sensitive.

The types of system in rows 3 and 4 of Table 4.1 have been defined in Section 4.4.4.1. Except for uncoupled wall systems of DC M, their q-factor includes explicitly an overstrength factor α_u/α_1 due to redundancy of the structural system. This is in addition to the factor of 1.5 due to overstrength of materials and elements (as in DC L), which is hidden in the DC M or H q-factor values. α_u/α_1 is the ratio of: a) the seismic action that turns the building into a full side-sway plastic mechanism, to b) the seismic action at formation of the first plastic hinge in the system (with the quasi-permanent gravity loads acting together with both these seismic actions); α_1 is the lowest value of $(M_{Rd}-M_V)/M_E$ among all members (M_{Rd} is the design value of moment resistance at the member end and M_E, M_V the bending moments there from elastic analysis for the design seismic action and the quasi-permanent gravity loads, respectively); α_u may be computed as the ratio of:

1. the seismic base shear causing a full plastic mechanism according to nonlinear static ("pushover") analysis per Section 3.3.2, to
2. the base shear due to the design seismic action.

For consistency with α_1, pushover analysis should use the design values, M_{Rd}, of moment resistance at member ends (Fig. 4.13).

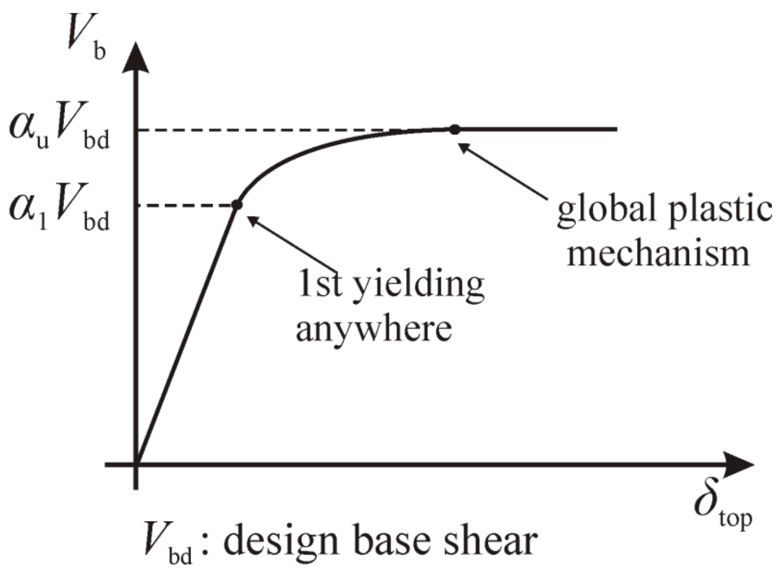


Fig. 4.13 Definition of facto3s α_u and α_1 on the basis of a base shear top displacement diagramme from pushover analysis

A practitioner is unlikely to carry out iterations of: a) pushover analyses and b) design based on elastic analysis, just to compute α_u/α_1 for the q-factor. So, Eurocode 8 gives default values of α_u/α_1.

For buildings regular in plan, the default values increase with the redundancy of the system, as follows:

– α_u/α_1 = 1.0 for wall systems with only two uncoupled walls per horizontal direction;

– α_u/α_1 = 1.1 for:

  - one-storey frame systems or frame-equivalent dual ones, and
  - wall systems with two or more uncoupled walls in the horizontal direction considered.

– α_u/α_1 = 1.2 for:

  - one-bay multi-storey frame systems and frame-equivalent dual ones,
  - wall-equivalent dual systems, and
  - coupled wall systems.

– α_u/α_1 = 1.3 for multi-storey multi-bay frames or frame-equivalent dual systems.

In a building which is irregular in plan per Eurocode 8 (see Section 4.3.2), the default value of α_u/α_1 is the average of:

– 1.0, and

– the default value given above for buildings regular in plan.

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

Buildings in rows 3 and 4 of Table 4.1 may use different q-factors in the two main horizontal directions, depending on the structural system and its vertical regularity or not in these two directions, but not by virtue of Ductility Class, which is the same for the entire building.

The relative magnitude of the values of q highlighted in the present Section reflect the position of Eurocode 8 on the effects of the type and regularity of the lateral-force-resisting-system on its earthquake resistance. This is an aspect to keep in mind during conceptual design.

Examples 4.10 to 4.12 at the very end of this Chapter illustrate some implications of the choice of Ductility Class, and of the corresponding value of the behaviour factor, for the design.

4.7 The option of "secondary seismic elements"

Eurocode 8, like other seismic codes, distinguishes the structural members which have a secondary role and contribution to earthquake resistance from the rest, calling them “secondary seismic” and “primary seismic” members, respectively (henceforth, “secondary" and “primary” members). The contribution of “secondary” members to the lateral stiffness and earthquake resistance of the building is not taken into account in the analysis for the seismic action. The building structure is considered to rely for its earthquake resistance only on “primary" members: “secondary” members are not considered as part of the lateral-load-resisting system.

Only “primary” members are designed and detailed for earthquake resistance following the rules of Eurocode 8. By contrast, “secondary” members follow the rules of Eurocode 2 and are fully considered and designed only for the non-seismic combinations of actions. The only requirement of Eurocode 8 on them is to maintain support of gravity loads under the most adverse displacements and deformations imposed on them in the seismic design situation, i.e., by the design seismic action and the concurrent gravity loads (see Section 5.9).

The designer is free to choose which members, if any, he/she may consider as “secondary”, subject to two restrictions introduced in Eurocode 8:

1. The total contribution to lateral stiffness of all “secondary” members may not exceed 15% of that of all “primary” ones.
2. The characterisation of some of members as “secondary” may not change the classification of the structure from irregular to regular. So:
   - if a frame, a column or a wall does not continue through the full height of the relevant part of the building, it cannot be classified as “secondary”;
   - if there is an abrupt change in the storey stiffness or (in infilled frame buildings) in the storey overstrength, this variation cannot be smoothened out by classifying some vertical elements as “secondary”;

– the eccentricity between any storey's centres of mass and stiffness may not be reduced from over 30% of the storey's torsional radius to less;

– the torsional radius in any direction may not increase from less than the radius of gyration of the masses to more by classifying some vertical elements as "secondary", etc.

The main reason to consider as "secondary" some of the members of a building designed for DC M or H is if they do not fall within the scope of Eurocode 8 for seismic design based on energy dissipation and ductility: flat slab frames and post-tensioned girders are prime examples. So, if the designer wants to use this type of concrete elements in a DC M or DC H building, he/she may have to rely for the seismic action only on walls or strong frames (usually along the perimeter), designating flat slabs, post-tensioned girders and their supporting columns as "secondary" members. As a matter of fact, in frame or frame-equivalent dual systems, columns supporting post-tensioned girders had better be taken as "secondary" anyway: normally the large size of prestressed girders makes it unfeasible to satisfy the strong-column/weak-beam capacity design rule, Eq. (5.31); moreover, such columns should have a cross-section sufficient for the support of gravity loads, but otherwise as small as feasible, in order to reduce the "parasitic" shears developing in these columns upon post-tensioning at the expense of the axial force in the girder.

The designer may also want to consider as "secondary" those members which – owing to architectural constraints – do not conform to the rules for geometry, dimensioning or detailing for energy dissipation and ductility, e.g. beams which:

– are connected to columns at an eccentricity violating Eq. (4.10), or

– are supported on columns which are not large enough to satisfy the Eurocode 8 rule for bond and maximum diameter of the top bars of the beam within the joint (see Section 5.2.3.3); or

– connect closely-spaced columns and hence develop a high seismic shear force (e.g., a large capacity-design shear from Eq. (5.42), owing to the short clear span, l_{cl}) that cannot be verified for the ULS in shear.

Unlike the cases which are outside the scope of Eurocode 8's design rules for energy dissipation and ductility, those of the paragraph above should preferably be accommodated through proper selection of the local structural layout, instead of resorting to “secondary” members. There are two good reasons for doing so:

- The earthquake “perceives” the structure as built, neither “knowing” much nor “caring” about the considerations and assumptions made in its design calculations. So, the “primary” members may perform well thanks to their ductility, but the “secondary” ones may suffer serious damage.
- A structural system that cannot be utilised in its entirety for the engineered earthquake resistance of the building is a waste of resources. This is more so, given the conservatism of the special design requirements for “secondary members” (see Sect. 5.9).

That said, the option of designing the entire structural system for strength instead of ductility (see Section 4.6.2.1) may be worth considering. In the framework of Eurocode 8, this means selecting DC L (Low) and q = 1.5. Then it is not necessary to make a distinction between “secondary” and “primary” members, as all members can be designed and detailed according to Eurocode 2, both for seismic and for non-seismic actions, without any regard to the special detailing and dimensioning rules of Eurocode 8 for energy dissipation and ductility.