

# **EARTHQUAKE-RESISTANT DESIGN OF CONCRETE BUILDINGS ACCORDING TO EN1998-1 (EUROCODE 8)**

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**ABSTRACT:** The key points of Part 1 of Eurocode 8 which are relevant to concrete buildings are overviewed. The paper covers the performance requirements and the provisions for the seismic action in EC8, the rules for buildings and the special rules for design and detailing of concrete buildings (including precast construction).

## **1 THE GENERAL PROVISIONS OF EN-EUROCODE 8 - PART 1 APPLICABLE TO CONCRETE STRUCTURES, ESPECIALLY BUILDINGS**

### **1.1 Introduction**

Although its main object is buildings, Part 1 of Eurocode 8 includes also the general provisions for the other parts of EC8 to build on: performance requirements, seismic action, analysis procedures, and general concepts and rules applicable to structures beyond buildings. It covers in separate sections the main structural materials – concrete, steel, composite (steel-concrete), timber and masonry. It also covers seismic design using base isolation.

The structure of Part 1 is the following:

1. General
2. Performance Requirements and Compliance Criteria
3. Ground Conditions and Seismic Action
4. Design of Buildings
5. Specific Rules for Concrete Buildings
6. Specific Rules for Steel Buildings
7. Specific Rules for Steel-Concrete Composite Buildings
8. Specific Rules for Timber Buildings
9. Specific Rules for Masonry Buildings
10. Base Isolation

In the following sections the key points of Part 1 of EC8 which are relevant to concrete buildings are overviewed.

### **1.2 Seismic action and performance requirements**

EC8 provides for a two-level seismic design, with the following explicit performance objective:

- Protection of life under a rare seismic action, by prevention of collapse of the structure or parts thereof and preservation of structural integrity and of residual load capacity.
- Reduction of property loss due to a frequent event, through limitation of structural and non-structural damage.

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 between 1.5 and 2 against substantial loss of gravity load capacity and lateral load resistance. The damage limitation performance level is achieved by limiting the overall deformations (lateral displacements) of the system to levels acceptable for the integrity of all its parts (including non-structural ones) and through (non-engineered) measures for the integrity of (masonry) infills.

Although not explicitly stated, another objective is to prevent global collapse during an extremely strong earthquake, of the order of the “Maximum Considered Earthquake” (MCE) of US codes (ATC 1997, BSSC 2000, SEAOC 1999) - although it is recognised that such collapse may be imminent afterwards and repair may be unfeasible or economically prohibitive. Tolerance to extremely strong

seismic actions is pursued through systematic and across-the-board application of the capacity design concept, to control the inelastic response mechanism.

Within the philosophy of national competence on issues of safety and economy, hazard levels corresponding to the two performance levels above are left for national determination. For structures of ordinary importance the recommendation of EC8 is for:

- A 10% exceedance probability in 50 years (“design”) seismic action for collapse prevention (mean return period: 475 years). The “design” seismic action for structures of ordinary importance over rock is termed “reference” seismic action.
- A 10% in 10 years “serviceability” action for damage limitation (mean return period: 95 years).

Enhanced performance of essential or large occupancy facilities is achieved not by upgrading the performance level for given earthquake level, as US codes do, but by modifying the hazard level (the mean return period) for which collapse prevention or damage limitation is pursued. For essential or large occupancy structures it is recommended to increase the seismic action at both performance levels, corresponding to an exceedance probability lower than 10% in 50 or 10 years, respectively. At the collapse prevention level the recommended value of the NDP-importance factor  $\gamma_I$  (to be applied to the reference seismic action) is 1.4 or 1.2 for essential or large occupancy buildings, respectively; a  $\gamma_I$  of 0.8 is also recommended for buildings of reduced importance for public safety.

The same spectral shape is meant to be used for the seismic action for damage limitation as for collapse prevention, with a universal multiplicative factor reflecting the difference in hazard level. The value of this factor should reflect national choice regarding protection of property, but also the regional seismotectonic environment. Values of 0.4 and 0.5 are indicated for this NDP-conversion factor, giving at the end approximately the same property protection for ordinary and large-occupancy buildings, 15-20% lower protection for buildings of low importance and 15% higher property protection for essential facilities (possibly allowing them to be operational during or immediately after a frequent event).

The elastic response spectrum of the “design” seismic action is anchored to the “reference” ground acceleration on rock,  $\alpha_{gR}$ , to be mapped in national zoning maps. The spectrum includes regions of constant spectral acceleration, pseudovelocity or displacement. The extent of each of these regions and spectral amplitudes therein depends on the regional seismotectonic environment and on soil type. The standard soil types are the following:

- Type A: rock, with a lower limit on the average shear wave velocity in the top 30m,  $v_s$ , of 800m/s;
- Type B: very dense sand or gravel, or very stiff clay, with  $v_s$  from 360 to 800m/sec;
- Type C: medium-dense sand or gravel, or stiff clay, with  $v_s$  from 180m/sec to 360m/sec;
- Type D: loose-to-medium sand or gravel, or soft-to-firm clay, with  $v_s$  less than 180m/sec;
- Type E: 5m to 20m thick soil with  $v_s$  less than 360m/sec, underlain by rock.

Definition of the spectral shapes for the standard soil types is left to National Annexes, depending on the magnitude of earthquakes contributing most to the hazard. In principle this allows a National Annex to introduce spectral amplification factors over each of the three regions of the spectrum – constant acceleration, pseudovelocity or displacement – which decrease with the absolute magnitude of the spectral value, due to the soil nonlinearity effect. This effect is more significant: a) for stronger ground motions, b) over soft soils and c) in the intermediate period range. This is the approach already taken in the USGS Seismic Hazard Maps from the USGS/BSSC 97 project (Frankel et al, 1996, 1997) and in all recent nationally applicable USA documents (e.g. ATC 1997, BSSC 2000). Records from the Loma Prieta earthquake, supplemented with laboratory and numerical results, has made possible quantification of the dependence of spectral amplification in the short and intermediate period ranges, on  $v_s$  and spectral acceleration level.

The entire elastic spectrum is anchored to the mapped “reference” acceleration on rock multiplied by: a) the importance factor  $\gamma_I$ ; b) a (NDP) factor  $S \geq 1$  reflecting the effect of soil conditions on spectral values; and c) a correction factor for damping  $\zeta$  other than 5%, equal to  $\sqrt{10/(5 + \zeta)}$ .

Instead of spectral amplification factors that decrease with increasing design acceleration (spectral or ground) as in US codes (ATC 1997, BSSC 2000), the (non-binding) recommendation of EC8 (in a

note) is for two types of spectra:

- Type 1 for moderate to large magnitude earthquakes;
- Type 2 for low magnitude ones (e.g. with surface magnitude less than 5.5) at close distances (producing over soft soils motions rich in high frequencies).

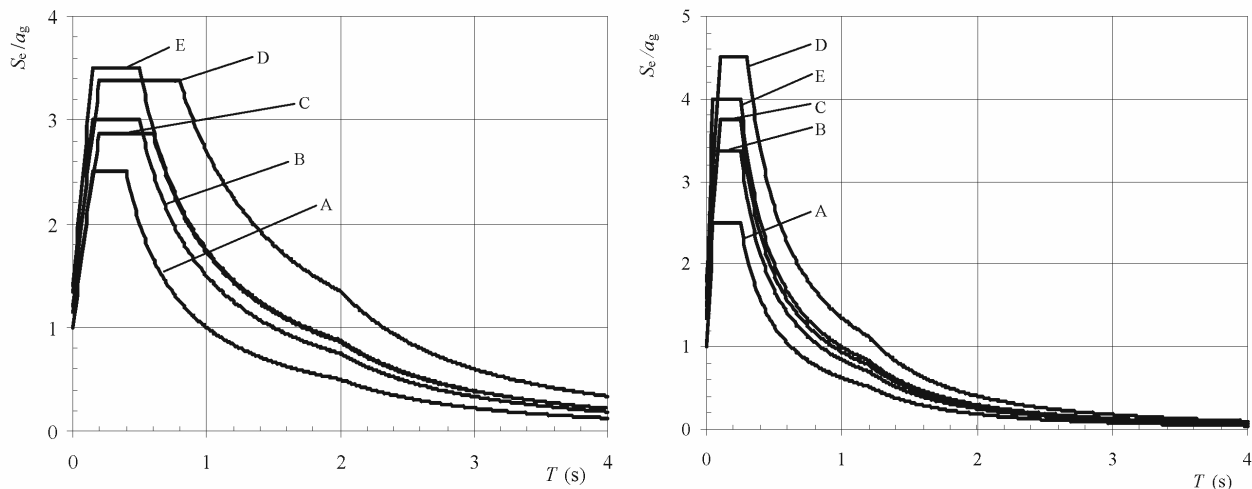


Figure 1: Elastic response spectra of Type 1 (left) and 2 (right) recommended in EC8 for the five standard ground types, PGA on rock 1g and 5% damping

The two types of recommended spectra are shown in Figure 1 for  $\zeta=5\%$ ,  $\gamma_I=1$  and PGA on rock of 1.0g. They were developed on the basis of a fairly large database of available records from the South of Europe and the Mediterranean area and are believed to be currently more representative of the seismic hazard of the region than the more refined description of elastic spectra in recent US codes. Noteworthy is the rapid decay of the spectra for long periods, the importance of which is limited though by the lower limit of 20% of acceleration at zero period recommended for design spectral accelerations (i.e. after division by the force reduction or behaviour factor,  $q$ ).

The seismic action section of EC8 gives also a detailed description of: a) the vertical elastic response spectrum; b) the peak ground displacement, and c) (in an informative annex) the displacement response spectrum, including the long period part. These items have been based on data from Europe, supplemented with information from other regions with similar seismotectonic features. They are recommended as function of soil type, type of spectrum (1 or 2) and acceleration of the horizontal component (including factors  $\gamma_I$  and  $S$ ).

EC8 does not have provisions for near-source effects; it provides though for topographic amplification (ridge effect, etc.) of the seismic action, with recommended amplification factors of 1.2, for isolated cliffs or long ridges with slope less than  $30^\circ$ , or of 1.4 for steeper ridges. Accounting for topographic amplification is mandatory for structures of importance above ordinary.

### 1.3 Classes of structures and design requirements

EC8 allows neglecting the seismic action in design, if it is considered so low that earthquake resistance is provided by the design for other actions. The recommended (as NDP) threshold is a value of the reference ground acceleration on top of the specific soil profile,  $S_{aGR}$ , times the importance factor,  $\gamma_I$ , equal to 0.05g.

EC8 also allows providing earthquake resistance by just dimensioning the members according to the other Eurocodes, for internal forces from an analysis for the seismic action with a force reduction or behaviour factor ( $q$ ) of 1.5 – considered available due to overstrength – without taking any other measures for local or global ductility. For non-base-isolated structures this simplification of design is recommended only for low seismicity regions. Although the designation of a region as a low seismicity one is left for the competent National Authorities as a NDP, a threshold value of 0.1g is

recommended for the product of  $\gamma_I$  and the design ground acceleration for the specific soil type ( $\gamma_I S \alpha_{gR}$ ). Building structures designed with just the material Eurocodes and a  $q$  value of 1.5 are termed “low-dissipative” or of Ductility Class (DC) low (L).

The majority of structures designed with EC8 are expected to be designed for “energy dissipation”. For buildings of the most common materials in earthquake-resistant construction, namely concrete, steel or composite (steel-concrete) buildings, two ductility classes (DCs) are provided for “dissipative” structures: Medium (M) and High (H) ductility. DC M and H buildings are entitled to values of the force reduction or behaviour factor  $q$  well above the minimum value of  $q=1.5$  attributed to overstrength. Availability of the global energy dissipation and ductility capacity needed for values of  $q$  (much) higher than 1.5 is ensured through:

- Measures to control the inelastic response mechanism, so that concentration of inelastic deformations in a part of the structure (mainly a soft storey mechanism) and brittle failure modes are avoided;
- Detailing of the plastic hinge regions for inelastic deformations expected to develop there under the design seismic action.

Concentration of inelastic deformations and soft storey mechanisms are avoided by configuring and dimensioning the lateral-force resisting system so that vertical members remain practically straight – i.e. elastic – above their base. Concrete wall (or dual) systems are promoted and their walls are (capacity-)designed to ensure that they remain elastic above the base. In concrete (but also in steel or composite) frames, columns are (capacity-)designed to be stronger than the beams, with an overstrength factor of 1.3 applied on beam design flexural capacities in their comparison with those of columns.

Once the global inelastic response mechanism is controlled, the force reduction or behaviour factor  $q$  of concrete buildings can be ultimately related to the local displacement and deformation demands on structural elements; e.g. through the global displacement ductility factor,  $\mu$ , and a  $q$ - $\mu$ - $T$  relation – the one by Vidic et al, 1994 underlies the relevant provisions of EC8. Detailing rules for elements aim at providing the required local deformation capacity.

The two upper Ductility Classes foreseen for concrete represent two different possible balances of strength and ductility, which are approximately equivalent in terms of total material cost and achieved performance under the design seismic action (i.e. at the collapse prevention level). DC M is slightly easier to design for and achieve at the construction site and may provide better performance in moderate earthquakes. DC H is believed to lead to enhanced performance under motions (much) stronger than the design seismic action. EC8 does not link selection between these two ductility classes to seismicity of the site or importance of the structure, nor puts any limit to their application. The choice is left to the National Annex, which may in turn leave it to the designer, depending on the particular project.

#### 1.4 Analysis procedures and models

Section 4 in Part 1 of EC8 provides the following analysis options for design and for evaluation of the performance of buildings:

- Linear static (termed “lateral force” method).
- Linear modal response spectrum analysis.
- Nonlinear static analysis (“pushover”).
- Nonlinear dynamic (response time-history).

Linear time-history analysis is not explicitly mentioned.

Unlike the US codes, where linear static analysis is the reference, in EC8 the linear modal response spectrum method is the standard procedure, applicable to all types of buildings.

The lateral force procedure may be applied if the effects of higher modes are not significant, i.e. only when:

- In both horizontal directions the fundamental period is less than 2sec and 4 times the transition period  $T_c$  between the constant-acceleration and the constant-velocity regions of the spectrum.
- There are no significant irregularities in elevation.

Regarding irregularity, unlike US codes (BSSC 2000, SEAOC 1999), which set quantitative – albeit arbitrary – criteria based on the heightwise distribution of storey mass, stiffness and strength, EC8 introduces qualitative criteria, easy to check at the preliminary design phase prior to any analysis:

- Lateral force-resisting systems continuous to the top of the – relevant part of the – building.
- Storey mass and stiffness that is constant or reduces gradually and smoothly to the top.
- Individual setbacks less than 10% of the underlying storey, but less than 30% of the ground floor dimension in total, if they are unsymmetric on the two sides of the building.
- – For frame buildings – smooth variation of overstrength of the individual storeys relative to the prescribed strength.

The lateral force method of analysis derives the base shear by assigning the total mass to the 1<sup>st</sup> translational mode in the horizontal direction of interest. It may use values of the fundamental period estimated through empirical expressions – the same as those adopted by the SEAOC '99 requirements (e.g.  $T_1=0.075H^{3/4}$  for RC frames,  $T_1=0.05H^{3/4}$  for RC wall buildings, with  $H$ =height from base in m). Such expressions represent lower (mean minus one standard deviation) bounds to values inferred from measurements on buildings in California in past earthquakes. As such, they are conservative, especially for structures of lower required earthquake resistance and hence less stiff. The use of a period calculated from mechanics – e.g. via the Rayleigh quotient – is allowed (as preferred in European design practice), without limiting the outcome relative to the empirical value. Moreover, if the fundamental period is less than twice the transition period  $T_c$  between the acceleration and the velocity-controlled spectral ranges, in buildings with more than two storeys the total lateral force may be reduced by 15% - to account for the difference between the 1<sup>st</sup> mode mass and the total. The objective of all these provisions is to bring the results of the lateral force method closer to those of response spectrum analysis, and not the other way around, as in US codes (BSSC 2000, SEAOC 1999). The total lateral force is distributed to the storeys following a 1<sup>st</sup> mode pattern of response accelerations, which may be taken as inverted triangular.

In the response spectrum analysis modal contributions are combined by rigorous application of the SRSS or CQC rules, i.e. at the level of the final seismic action effects of interest (internal forces, displacements, etc.). This is different from the US approach, where the modal response spectrum analysis procedure is cast to look like linear static analysis, in that storey lateral forces are calculated for each mode (from modal storey accelerations) and combined via the SRSS (or CQC) rule, for the structural system to be analysed for the resulting lateral forces as in the linear static procedure. (A disadvantage of the theoretically sound EC8 approach of applying the SRSS or CQC rules to the modal internal forces is that signs are lost and physical meaning as well).

Pushover analyses should be performed under two lateral load patterns: one corresponding to uniform lateral accelerations and another similar to the lateral forces used in linear static (lateral force) analysis, if applicable, or derived from a modal response spectrum one. The target displacement for pushover analysis is defined in an informative annex according to the N2 procedure by Fajfar (2000).

Nonlinear response-history analysis should use as input at least three artificial, recorded, or simulated records (or pairs of different records, for analysis in 3D), the mean elastic spectrum of which should no-where fall below that of the design seismic action by more than 10%. The results of nonlinear dynamic analyses are averaged, if at least seven such analyses are performed; otherwise the most unfavourable results in the analyses performed are used.

In buildings which are regular in plan, two independent 2D models may be used for the analysis of the response to the two horizontal components of the seismic action. A building may be identified as regular-in-plan, prior to any analysis, if it has:

- Rigid diaphragms, nearly rectangular in plan (re-entrant corners reducing floor area by not more than 5% each), with aspect ratio less than 4.

- Eccentricity between the storey centres of mass and stiffness less than 30% of the corresponding torsional radius (square root of ratio of torsional to lateral stiffness, with stiffness parameters estimated in most cases from the moments of inertia of vertical elements), and
- Torsional radius less than the radius of gyration of the floor plan.

Two separate 2D models may also be used for buildings of ordinary importance with: a) height less than 10m or 40% of the plan dimensions b) storey centres of mass and stiffness approximately on (two) vertical lines, and c) in both horizontal directions torsional radius not less than the SRSS of the radius of gyration of the floor in plan and of the projection of the eccentricity between centers of mass and stiffness in that direction. If conditions a) and b) are fulfilled, but not condition c), then two separate 2D models may still be used, provided that all seismic action effects from the 2D analyses are increased by 25%.

Regardless of whether they are computed via a single 3D or two separate 2D models, seismic action effects due to the individual horizontal components are combined through the SRSS rule or the 1:0.3 rule. Maximum values of action effects estimated individually through the SRSS rule may be conservatively assumed to take place at the same time. Nonetheless, more accurate and less conservative rules may be introduced by the National Annex for the estimation of the likely simultaneous values of action effects due to the different components of the seismic action. In buildings regular in plan, with independent lateral-force-resisting systems in the two directions consisting solely of walls or bracing systems, the effects of the two horizontal components do not need to be combined.

Accidental eccentricity is taken equal to 5% of the perpendicular plan dimension, without amplification due to torsional-lateral coupling. Its effects may be calculated statically, by applying storey torsional moments to a 3D structural model, even when the modal response spectrum method is used for the analysis of the response to the two horizontal components. The effects of accidental eccentricity may be accounted for in a simpler and conservative way by amplifying the results of the – linear static or dynamic, or nonlinear static – analysis for each horizontal component by  $1+0.6x/L$ , with  $x$  denoting distance of the element of interest from the centre in plan and  $L$  the plan dimension, both normal to the direction of the seismic action. This factor is derived assuming that torsional effects are fully resisted by the lateral-force-resisting-elements in the horizontal direction considered and that these elements are uniformly distributed in plan. In structures regular in plan and analysed with two separate 2D models, factor 0.6 is replaced by 1.2.

Amplification of eccentricities between centres of mass and stiffness is not required. This is convenient, as in most cases storey stiffness centres cannot be uniquely defined; moreover, determination of their position at a level of accuracy and sophistication consistent with the dynamic amplification of natural eccentricity, requires tedious additional analyses.

In all types of analysis the mathematical model should include only the elements considered as part of the lateral-force-resisting-system, termed “primary” elements. Nonetheless, elements not considered as part of the lateral-force-resisting system should collectively account for less than 15% of total lateral stiffness. Moreover, they should be distributed regularly and uniformly in plan and elevation, so that they don’t affect the regularity classification of the structural system.

The elastic stiffness used in linear analysis should be the secant stiffness to yielding; it may be taken as half of the uncracked stiffness of the gross concrete section. Nonlinear analysis may use this value as pre-yield stiffness (pre- and post cracking stiffnesses may be considered also, if so-desired), and may neglect the effect of strain hardening on post-yield stiffness. (Post-yield stiffness should be taken negative, if significant strength degradation develops).

The requirement on hysteresis rules to be used in nonlinear response-history analysis is just to reflect realistically energy dissipation within the expected range of displacement amplitudes. Nonlinear element models should be based on mean values of material properties, which are higher than nominal values.

Displacements from linear analysis are calculated in general on the basis of the equal displacement

rule, i.e. by multiplying elastic analysis results by the  $q$ -factor. Allowance is made for more accurate calculation, including the  $q$ - $\mu$ - $T$  relation by Vidic et al, 1994, given in the informative annex for the determination of the target displacement for pushover analysis. Resulting displacements are larger than those calculated according to the rules of US codes (BSSC 2000, SEAOC 1999). Storey drifts determined in this way are used for the estimation of  $P$ - $\Delta$  effects as a ratio to 1<sup>st</sup>-order ones. If the ratio  $\theta = P\delta/V_h$  exceeds 0.1 in any storey, 1<sup>st</sup>-order analysis results are divided by  $1-\theta$ .  $P$ - $\Delta$  effects are calculated on the basis of the secant-to-peak-drift storey stiffness, instead of the elastic stiffness used in US codes. Therefore, their magnitude influences analysis results more often and to a larger extent than according to US codes.

Non-engineered masonry infills producing irregularities in plan or elevation should be taken into account in the analysis of concrete buildings of DC H, unless walls provide at least 50% of the lateral force resistance. (The same requirement applies to steel or composite moment frames of DC H). Irregular distribution of infills in plan may normally be considered by doubling the effects of accidental eccentricity. Extremely irregular arrangements, as e.g. when infills are concentrated mainly along one or two sides of the perimeter, require instead a 3D model explicitly including the infills, along with a sensitivity analysis of the effect of their properties and position (including removal of one out of 3 or 4 infill panels from the model). Attention is called to the possible effects on structural elements furthest away from the sides of the plan where infills may be concentrated. A reduction in the amount of infills relative to the storey above (as e.g. in open ground-storeys) should be taken into account, by amplifying seismic action effects from the analysis, so that the deficit in infill shear strength in that storey is fully compensated by an increase in resistance of the frame (vertical) members there.

## 4 DIMENSIONING AND DETAILING IN CONCRETE BUILDINGS

### 2.1 Introduction

The standard procedure in EC8 is force-based design on the basis of the results of linear elastic analysis (static with lateral forces, or modal response spectrum), for the elastic spectrum reduced by the behaviour factor  $q$ . In buildings designed for energy dissipation (i.e. those of ductility classes M and H), design also aims at controlled inelastic response, by preventing storey-sway mechanisms and brittle failure modes through capacity design of members, and by detailing regions intended for energy dissipation (plastic hinge or “critical” regions) for ductility and deformation capacity.

EC8 allows also design on the basis of nonlinear analysis, static (pushover) or dynamic (response-history), with verification of members by comparing directly deformation supplies and demands. Fairly detailed rules are given in EC8 for the calculation of deformation demands through nonlinear analysis. On the supply side, the definition of acceptable deformation limits for members is left to National Annexes, in the form of supplementary non-contradictory information. To ensure a minimum of global and local ductility for the so-designed buildings, EC8 requires that they meet all DCM rules for member detailing, strong columns-weak beams in frames and capacity design in shear.

In the following, the EC8 provisions for the standard procedure of force-based design on the basis of an elastic spectrum reduced by a  $q$ -factor are summarized.

### 2.2 Behaviour factor $q$ for reduction of elastic forces

For structures designed for energy dissipation the behaviour factor,  $q$ , by which the elastic spectrum for use in linear analysis is reduced, is linked, directly or indirectly, to the ductility and deformation demands on members and connections, and hence to the corresponding detailing requirements. Its value depends on the type of lateral-force-resisting-system and on the ductility class selected for the design.

System overstrength is explicitly included in the value of the  $q$ -factor through the ratio  $\alpha_w/\alpha_1$  (denoted here for convenience  $\alpha_R$ ) of the seismic action that causes development of a full plastic mechanism

(i.e. for fully yielded structure) to the seismic action at development of the first plastic hinge in the system – both in the presence of the gravity loads considered to act simultaneously with the seismic action. This ratio reflects system overstrength due to redundancy. EC8 gives default values for it in buildings regular in plan, as follows:

- 1.1 for one-storey frames and wall systems with more than two uncoupled walls per direction;
- 1.2 for one-bay multistorey frames, or dual systems with walls providing 50% to 65% of lateral force resistance or coupled wall systems;
- 1.3 for multistorey multi-bay frames.

In buildings which are not regular in plan, the default value of the overstrength factor is the average of (a) 1.0 and (b) the default value quoted above for buildings regular in plan.

Higher values may be used, up to a maximum value of 1.5, provided that they are confirmed through a pushover analysis of the so-designed structure.

The  $q$ -factors for concrete frames, dual systems with walls providing up to 65% of lateral force resistance) and coupled-wall systems (i.e. with at least 65% of the lateral force resistance provided by walls, and at least half of the wall resistance provided by coupled walls - i.e. by walls in which coupling reduces the sum of individual wall base moments by at least 25%) are:  $q=3\alpha_R$  for DC M and  $4.5\alpha_R$  for DC H.

Systems of uncoupled walls (i.e. systems with at least 65% of the lateral force resistance provided by walls, but with less than half of the wall resistance provided by coupled walls) are entitled to  $q=3$  for DCM and  $q=4\alpha_R$  for DC H.

Torsionally sensitive buildings (defined as those with lateral-force-resisting system concentrated near the centre in plan, so that the torsional radius in at least one horizontal direction is less than the radius of gyration of the floor plan) are exempted and penalized with  $q=2$  for DC M, or 3 for DC H.

“Inverted pendulum” systems (defined as those with at least 50% of mass in the upper third of the height, or with energy dissipation at the base of a single element) are penalized also with  $q=1.5$  for DC M or 2 for DC H (one-storey buildings on columns with axial load ratio  $v_d < 0.3$  are not classified as inverted pendulum systems).

The values of  $q$  quoted above are called basic values,  $q_0$ , of the  $q$ -factor; they are the ones to be used for the estimation of ductility demands on element regions intended for inelastic energy dissipation and detailed accordingly. For the purposes of calculation of seismic action effects from linear analysis, the basic values of  $q$  are modified as follows:

- In buildings with concrete walls providing more than 50% of the lateral force resistance, the basic value  $q_0$  is multiplied by a factor equal to  $(1+\alpha_o)/3$  with values between 0.5 and 1, where  $\alpha_o$  denotes the (mean) wall aspect ratio (sum of wall heights,  $H_w$ , divided by sum of wall cross-sectional lengths,  $l_w$ ).
- In buildings with vertical irregularities, the  $q$ -factor value is reduced by 20%.

Regardless of such reductions, DC M and H structures are entitled a  $q$ -factor of 1.5, considered to be always available owing to overstrength alone.

A building may have different  $q$ -factors in the two main horizontal directions, depending on the structural system and its vertical regularity classification in the two directions, but not due to Ductility Class, which should be the same for the entire building.

### 2.3 Verification and detailing of members

Except when most of the lateral force resistance is provided by concrete walls, member sizes will be controlled by the limit on storey drift ratio under the frequent seismic action calculated for this seismic action on the basis of the equal-displacement rule. This limit is equal to 0.5%, if the non-structural elements are brittle and attached to the structure, or to 0.75% if they are ductile or not forced to follow structural deformations. It will be up to the national annex to set the level of the “serviceability”



earthquake and hence determine to which extent these limits will control design; nonetheless, with the values of 0.4 to 0.56 recommended for the factor to be applied on the design seismic action for conversion to the “serviceability” one, these limits are from 2 to 3 times more restrictive than those of US codes (BSSC 2000, SEAOC 1999). Given the large global stiffness necessary to meet storey drift limits, the limits on the coefficient  $\theta$  for P- $\Delta$  effects (an upper limit of 0.3, with geometrically nonlinear analysis required for  $\theta > 0.2$ ) are not critical for buildings.

In infilled frames of any ductility class and unless walls provide at least 50% of the lateral force resistance, additional non-engineered measures are specified to control damage to infills.

In DC M and H structures, elements where inelastic action is intended to develop are dimensioned for the ULS against ductile, “deformation controlled”, failure modes for the seismic action effects from the analysis. This applies primarily to bending of beams in moment frames and to bending at the base section of ductile concrete walls and of columns in moment frames. In frames, dimensioning for the ULS in bending on the basis of the analysis results alone applies also to the top-storey columns, to one-out-of-four columns in a frame and to all storeys of columns in systems where at least 50% of lateral force resistance is provided by walls. (In all other cases column flexural capacity may be conditioned by the strong column-weak beam rule). Dead loads enter in this verification with the nominal value and live loads (if unfavourable) with their quasi-permanent arbitrary-point-in-time value, which for residential and office buildings is 30% of the nominal and for public areas 60% of it.

As in concrete a shear failure mode is considered brittle, the design shear force is derived from capacity design in the members of DC M and DC H buildings and in joints of DC H frames. Capacity design shear in concrete beams and columns is based on the flexural capacities of the most critical members around a joint (e.g. if plastic hinges form in beams, the capacity design shear of the column is based on the beam flexural capacities) and includes an overstrength factor between 1.0 and or 1.3. Capacity design shears in DC H beam-column joints are always based on the capacities of beam bars crossing the joint with an overstrength factor of 1.2.

Local deformation and ductility demands to be sustained by those members or parts thereof intended to develop inelastic action, increase as the DC (and with it the value of  $q$ ) increases. Deformation and ductility capacity is a function of the inherent ductility of the materials and of the configuration and detailing of the members. So, ductility requirements on materials increase with the DC: the nominal cylindrical concrete strength should be above 16 MPa for DC M, or 20 MPa for DC H: Reinforcing steel of DC L or M should have a hardening ratio,  $f_t/f_y$ , of at least 1.08 and elongation at maximum stress,  $\epsilon_u$ , at least 5% (both values refer to lower 10% fractiles). For DC H,  $\epsilon_u$  should be at least 7.5%,  $f_t/f_y$  should be between 1.15 and 1.35 and the upper 5% fractile of the actual yield stress,  $f_{yk,0.95}$ , should not exceed the nominal yield strength by more than 25%. The upper limits aim at controlling flexural overstrength.

Tables 1 to 3 summarize the main detailing and dimensioning rules in EC8, for beams, columns and walls of the three ductility classes. Prescriptive detailing rules among them are overall slightly stricter than those provided by US codes (BSSC 2000, SEAOC 1999) for the corresponding DC (with “Intermediate” corresponding to M and “Special” to H). Rules for anchorage of beam bars at or through beam-column joints in Table 3 are more detailed and more demanding than in US codes.

Detailing rules controlling the deformation capacity of plastic hinges in concrete members, by controlling confining reinforcement in columns or boundary elements of walls (in Tables 2 and 3) and the compression reinforcement in beam end sections (see Table 1), are linked through a simple analytical procedure to the local curvature ductility factor,  $\mu_\phi$ , and through it to the basic value of the behaviour factor:

$$\mu_\phi = 2q_o - 1 \quad \text{if } T_1 \geq T_C \quad (1)$$

$$\mu_\phi = 1 + 2(q_o - 1)T_C/T_1 \quad \text{if } T_1 < T_C \quad (2)$$

where  $T_1$  the fundamental period of the building and  $T_C$  the period at the upper limit of the constant acceleration region of the spectrum. Eqs. (1), (2) are based on a relation between  $\mu_\phi$  and the

displacement ductility factor,  $\mu_\delta$ :  $\mu_\phi=2\mu_\delta-1$ , which is normally a conservative approximation for concrete members, and on the Vidic et al (1994) relation between  $\mu_\delta$  and  $q$ :

$$\mu_\delta=q \text{ if } T_1 \geq T_C, \quad \mu_\delta=1+(q-1)T_C/T_1 \text{ if } T_1 < T_C \quad (3)$$

So the continuous spectrum of  $q$ -values resulting from the introduction of the overstrength factor  $\alpha_R$ , produces detailing that also varies continuously.

Rules for confinement of column end regions do not apply uniformly to all column ends, but only to those intended for plastic hinge development. In end regions of columns protected from plastic hinging through capacity design in flexure at beam/column joints, only the prescriptive rules – e.g. against buckling of rebars, etc. – do apply. Specially for DC H concrete buildings, confinement requirements in such regions are reduced to 2/3 of those applied where plastic hinges are expected.

Dimensioning rules for DC H buildings cover special aspects, such as:

- Inclined reinforcement in two directions in beams other than coupling beams, subjected to large shear reversals (see bottom of Table 1).
- Inclined reinforcement in walls against sliding at construction joints (see last part of Table 3).
- Explicit verification of joints in shear and dimensioning of their reinforcement.

The EC8 rules for concrete elements not considered as part of the lateral-force-resisting system (“secondary” elements) require that the displacements imposed on them do not drive them beyond their design moment and shear capacities calculated according to the rules of EC2.

#### 1.4 Provisions for walls

The use of concrete walls is promoted in EC8, through:

- the low drift limits, which are difficult to satisfy with concrete frames alone – especially as the cracked stiffness of concrete members is used,
- the high  $q$ -factors provided for dual and wall systems, and
- the exemption of columns from capacity design in flexure at beam-column joints, when walls resist at least 50% of the lateral force.

To fully exploit their potential in preventing soft storey mechanisms, ductile concrete walls should be designed and detailed not only for a large plastic hinge rotation at the base, but also against premature flexural yielding or shear failure elsewhere along their height. To this end ductile DC M and H walls are dimensioned (see Table 5):

- In flexure for a heightwise linear envelope of their moment diagram from the analysis,
- In shear for a shear diagram resulting from amplification of that from the analysis in the lower third of the wall height and then proceeding linearly to a shear at the top not less than half the amplified shear at the base. The amplification of shears at the base is by 50% in DC M walls and (much) higher in DC H ones, depending on the flexural overstrength at the base and the fundamental period of the building.

This overdesign of ductile walls in bending and shear protects them against higher mode effects that develop after flexural yielding at the base and are not detected by elastic analysis.

EC8 includes special provisions for systems consisting of a fairly large number of large but lightly reinforced concrete walls, founded either individually or jointly on common foundation elements (e.g. a raft). To qualify for these special design provisions of EC8, such a system should have in each direction at least two walls (for redundancy and torsional resistance; one wall in one of the two directions is sufficient, if the  $q$  factor is reduced by 50%), supporting together at least 20% of the total gravity load (i.e. at least 40% in total for the walls of the two directions), with cross-sectional length  $l_w$  of at least 4m (or 2/3 of the total height for building less than 6m tall). Due to their large dimensions, or lack of fixity at the base, or connection with other large walls in the orthogonal direction, these walls cannot develop large rotations at a flexural plastic hinge at the base. They will develop, instead, rigid-body rocking rotations in a single- or multi-rigid-block fashion, at the foundation if they are on (not tied-down) spread footings, or at discrete horizontal cracks at their base or at floor levels. Due to

the large cross-sectional length of these walls, such rotations induce significant uplift of the centroid of the section, raising the tributary gravity load and the ends of beams framing into the wall, both to the benefit of the global response and stability of the system. These systems cannot be meaningfully designed and detailed for ductile response based on development of a single flexural hinge at the base. Instead, and provided their large walls resist at least 65% of the lateral force on each direction, they can be designed as DC M wall systems with  $q=3$  and the following special provisions:

- They are dimensioned for the ULS in flexure without amplification of the moments from the analysis above the base and without excess vertical reinforcement – e.g. with less minimum web vertical reinforcement than normally required for walls. So discrete cracking and flexural yielding may take place at several horizontal sections and pre-emptive shear failure due to flexural overstrengths will be avoided. An additional axial force of at least half of that due to gravity loads is considered in this ULS verification, to account for vertical vibrations set up by impact at closing cracks or at the foundation.
- To preclude shear failure, the wall is dimensioned in shear for a shear force about twice that from the analysis (i.e. much higher than in ductile walls of DC M). In return, and due to the transient and deformation-controlled nature of the response, if this verification in shear shows no need of shear reinforcement, the minimum amount of smeared horizontal reinforcement normally required in walls may not be placed.

### 1.5 Verification of foundation and design and detailing of foundation elements

In DC M and H structures the verification of the foundation soil and – in general – dimensioning of foundation elements, are also based on seismic action effects derived from capacity design. This is in the opposite direction than that in US codes (BSSC 2000, SEAOC 1999) – which allow reduction of overturning moment at the base due to uplift by 25% for linear static analysis or by 10% for a response spectrum one – and is due to the importance of the foundation for the integrity of the whole and the difficulty to access, inspect and repair damaged foundation systems.

Capacity design seismic effects, to be superimposed to those due to gravity loads, are the – factored by 1.2 – seismic action effects which occur at the intended plastification of the component of the superstructure with the highest influence on the foundation element in question (i.e. the base of a frame column or wall in the direction which yields first in bending). For common foundations of more than one vertical element (e.g. rafts, foundation beams, etc.) plastification in the element with the largest design seismic shear is considered in these capacity-design calculations. Alternatively, the seismic action effects from the analysis multiplied by 1.4 may be used for common foundation elements.

Foundation elements (including piles) may be designed for energy dissipation in them, by being dimensioned for seismic action effects derived from the analysis, instead of the above capacity design calculations. In that case they should meet all the dimensioning and detailing rules of the corresponding DC for elements of the superstructure (e.g. capacity design in shear for foundation- or tie-beams). If, instead, they are dimensioned for the above capacity-design effects, they are subject to the rules of DC L structures, i.e. design according to EC2 alone.

### 1.6 Precast concrete systems

The provisions of EC8 for precast structures strive for design with emulation of monolithic construction. Emulation may be based either on flexural yielding at “energy dissipating” (“ductile”) connections between precast elements, or on “overdesigned” (“strong”) connections designed to remain elastic while flexural yielding develops elsewhere. Design with elements interconnected at dry joints is not covered; for such systems use may be made of the facility of experimental demonstration of the cyclic deformation capacity and energy dissipation of “energy dissipating connections” not complying with the set of EC8 rules for precast structures.

In recognition of possible differences in precast systems produced and marketed in different Member States, EC8 allows fixing their  $q$ -factor value in National Annexes as NDP. The EC8 recommendation

is that precast structures may use the same DCs and q-factors as monolithic ones, provided their connections comply with all the rules of monolithic structures, or are capacity-designed to remain elastic, or are demonstrated experimentally to be equivalent to monolithic connections. Otherwise, it is recommended to apply a reduction of 50% to the q-factor applicable for monolithic structures. In all cases, though, a precast concrete system is entitled to a q-factor value of at least 1.5, just owing to overstrength.

## REFERENCES

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Table 1: EC8 rules for detailing and dimensioning of primary beams (secondary beams: as in DCL)

	DC H	DCM	DCL
“critical region” length	1.5h <sub>w</sub>	h <sub>w</sub>	
<i>Longitudinal bars (L):</i>			
ρ <sub>min</sub> , tension side	0.5f <sub>ctm</sub> /f <sub>yk</sub>		0.26f <sub>ctm</sub> /f <sub>yk</sub> , 0.13% <sup>(0)</sup>
ρ <sub>max</sub> , critical regions <sup>(1)</sup>	ρ' + 0.0018f <sub>cd</sub> /(μ <sub>φ</sub> ε <sub>sy,d</sub> f <sub>yd</sub> ) <sup>(1)</sup>		0.04
A <sub>s,min</sub> , top & bottom	2Φ14 (328mm <sup>2</sup> )	-	
A <sub>s,min</sub> , top-span	A <sub>s,top-supports</sub> /4	-	
A <sub>s,min</sub> , critical regions bottom	0.5A <sub>s,top</sub> <sup>(2)</sup>		-
A <sub>s,min</sub> , supports bottom	A <sub>s,bottom-span</sub> /4 <sup>(0)</sup>		
d <sub>bL</sub> /h <sub>c</sub> - bar crossing interior joint <sup>(3)</sup>	$\leq \frac{6.25(1+0.8v_d)}{(1+0.75\frac{\rho'}{\rho_{max}})} \frac{f_{ctm}}{f_{yd}}$	$\leq \frac{7.5(1+0.8v_d)}{(1+0.5\frac{\rho'}{\rho_{max}})} \frac{f_{ctm}}{f_{yd}}$	-
d <sub>bL</sub> /h <sub>c</sub> - bar anchored at exterior joint <sup>(3)</sup>	$\leq 6.25(1+0.8v_d) \frac{f_{ctm}}{f_{yd}}$	$\leq 7.5(1+0.8v_d) \frac{f_{ctm}}{f_{yd}}$	-
<i>Transverse bars (w):</i>			
(i) outside critical regions			
spacing s <sub>w</sub> ≤	0.75d		
ρ <sub>w</sub> ≥	0.08√(f <sub>ck</sub> (MPa)/f <sub>yk</sub> (MPa)) <sup>(0)</sup>		
(ii) in critical regions:			
d <sub>bw</sub> ≥	6mm		
spacing s <sub>w</sub> ≤	6d <sub>bL</sub> , $\frac{h_w}{4}$ , 24d <sub>bw</sub> , 175mm	8d <sub>bL</sub> , $\frac{h_w}{4}$ , 24d <sub>bw</sub> , 225mm	-
<i>Shear design:</i>			
V <sub>Ed</sub> , seismic <sup>(4)</sup>	$1.2 \frac{\sum M_{Rb}}{l_{cl}} \pm V_{o,g+\psi_2q}$ <sup>(4)</sup>	$\frac{\sum M_{Rb}}{l_{cl}} \pm V_{o,g+\psi_2q}$ <sup>(4)</sup>	From the analysis for the “seismic design situation”
V <sub>Rd,max</sub> seismic <sup>(5)</sup>	As in EC2: V <sub>Rd,max</sub> = 0.3(1-f <sub>ck</sub> (MPa)/250)b <sub>w0</sub> z <sub>fcd</sub> sin2δ <sup>(5)</sup> , with 1 ≤ cotδ ≤ 2.5		
V <sub>Rd,s</sub> , outside critical regions <sup>(5)</sup>	As in EC2: V <sub>Rd,s</sub> = b <sub>w</sub> zρ <sub>w</sub> f <sub>ywd</sub> cotδ <sup>(5)</sup> , with 1 ≤ cotδ ≤ 2.5		
V <sub>Rd,s</sub> , critical regions <sup>(5)</sup>	V <sub>Rd,s</sub> = b <sub>w</sub> zρ <sub>w</sub> f <sub>ywd</sub> (δ = 45°)	As in EC2: V <sub>Rd,s</sub> = b <sub>w</sub> zρ <sub>w</sub> f <sub>ywd</sub> cotδ, with 1 ≤ cotδ ≤ 2.5	
If ζ ≡ V <sub>Emin</sub> /V <sub>Emax</sub> <sup>(6)</sup> < -0.5: inclined bars at angle ±α to beam axis, with cross-section A <sub>s</sub> /direction	If V <sub>Emax</sub> /(2+ζ)f <sub>ctd</sub> b <sub>w</sub> d > 1: A <sub>s</sub> = 0.5V <sub>Emax</sub> /f <sub>yd</sub> sinα & stirrups for 0.5V <sub>Emax</sub>		-

- (0) NDP (Nationally Determined Parameter) according to Eurocode 2. The Table gives the value recommended in Eurocode 2.
- (1) μ<sub>φ</sub> is the value of the curvature ductility factor that corresponds to the basic value, q<sub>0</sub>, of the behaviour factor used in the design.
- (2) The minimum area of bottom steel, A<sub>s,min</sub>, is in addition to any compression steel that may be needed for the verification of the end section for the ULS in bending under the (absolutely) maximum negative (hogging) moment from the analysis for the “seismic design situation”, M<sub>Ed</sub>.
- (3) h<sub>c</sub> is the column depth in the direction of the bar, v<sub>d</sub> = N<sub>Ed</sub>/A<sub>c</sub>f<sub>cd</sub> is the column axial load ratio, for the algebraically minimum value of the axial load in the “seismic design situation”, with compression taken as positive.
- (4) At a member end where the moment capacities around the joint satisfy: ΣM<sub>Rb</sub> > ΣM<sub>Rc</sub>, M<sub>Rb</sub> is replaced in the calculation of the design shear force, V<sub>Ed</sub>, by M<sub>Rb</sub>(ΣM<sub>Rc</sub>/ΣM<sub>Rb</sub>)
- (5) z is the internal lever arm, taken equal to 0.9d or to the distance between the tension and the compression reinforcement, d-d<sub>1</sub>.
- (6) V<sub>Emax</sub>, V<sub>Emin</sub> are the algebraically maximum and minimum values of V<sub>Ed</sub> resulting from the ± sign; V<sub>Emax</sub> is the absolutely largest of the two values, and is taken positive in the calculation of ζ; the sign of V<sub>Emin</sub> is determined according to whether it is the same as that of V<sub>Emax</sub> or not.

Table 2: EC8 rules for detailing and dimensioning of primary columns (secondary columns as in DCL)

	DCH	DCM	DCL
Cross-section sides, $h_c, b_c \geq$	0.25m; $h_v/10$ if $\theta=P\delta/Vh>0.1^{(1)}$	-	-
“critical region” length $^{(1)} \geq$	$1.5h_c, 1.5b_c, 0.6m, l_c/5$	$h_c, b_c, 0.45m, l_c/5$	$h_c, b_c$
<i>Longitudinal bars (L):</i>			
$\rho_{min}$	1%		$0.1N_d/A_c f_{yd}, 0.2\%^{(0)}$
$\rho_{max}$	4%		$4\%^{(0)}$
$d_{bl} \geq$	8mm		
bars per side $\geq$	3		2
Spacing between restrained bars	$\leq 150mm$	$\leq 200mm$	-
distance of unrestrained bar from nearest restrained bar	$\leq 150mm$		
<i>Transverse bars (w):</i>			
Outside critical regions:			
$d_{bw} \geq$	6mm, $d_{bl}/4$		
spacing $s_w \leq$	$20d_{bl}, h_c, b_c, 400mm$	$12d_{bl}, 0.6h_c, 0.6b_c, 240mm$	
at lap splices, if $d_{bl} > 14mm$ : $s_w \leq$	$12d_{bl}, 0.6h_c, 0.6b_c, 240mm$		
Within critical regions: <sup>(2)</sup>			
$d_{bw} \geq^{(3)}$	$6mm, 0.4(f_{yd}/f_{ywd})^{1/2}d_{bl}$	6mm, $d_{bl}/4$	
$s_w \leq^{(3),(4)}$	$6d_{bl}, b_o/3, 125mm$	$8d_{bl}, b_o/2, 175mm$	-
$\omega_{wd} \geq^{(5)}$	0.08	-	
$\alpha\omega_{wd} \geq^{(4),(5),(6),(7)}$	$30\mu_\phi^* v_d \varepsilon_{sv,d} b_c/b_o - 0.035$	-	
In critical region at column base:			
$\omega_{wd} \geq$	0.12	0.08	-
$\alpha\omega_{wd} \geq^{(4),(5),(6),(8),(9)}$	$30\mu_\phi v_d \varepsilon_{sv,d} b_c/b_o - 0.035$		-
Capacity design check at beam-column joints: <sup>(10)</sup>	$1.3\sum M_{Rb} \leq \sum M_{Rc}$ No moment in transverse direction of column		-
Verification for $M_x-M_y-N$ :	Truly biaxial, or uniaxial with $(M_z/0.7, N), (M_y/0.7, N)$		
Axial load ratio $v_d = N_{Ed}/A_c f_{cd}$	$\leq 0.55$	$\leq 0.65$	-
<i>Shear design:</i>			
$V_{Ed}$ seismic <sup>(11)</sup>	$1.3 \frac{\sum M_{Rc}^{ends}}{l_{cl}}^{(11)}$	$1.1 \frac{\sum M_{Rc}^{ends}}{l_{cl}}^{(11)}$	From the analysis for the “seismic design situation”
$V_{Rd,max}$ seismic <sup>(12), (13)</sup>	As in EC2: $V_{Rd,max} = 0.3(1-f_{ck}(MPa)/250)b_w z f_{cd} \sin 2\delta$ , with $1 \leq \cot \delta \leq 2.5$		
$V_{Rd,s}$ seismic <sup>(12), (13), (14)</sup>	As in EC2: $V_{Rd,s} = b_w z \rho_w f_{ywd} \cot \delta + N_{Ed}(h-x)/l_{cl}^{(13)}$ with $1 \leq \cot \delta \leq 2.5$		

- (0) Note (0) of Table 1 applies.
- (1)  $h_v$  is the distance of the inflection point to the column end further away, for bending within a plane parallel to the side of interest;  $l_c$  is the column clear length.
- (2) For DCM: If a value of  $q$  not greater than 2 is used for the design, the transverse reinforcement in critical regions of columns with axial load ratio  $v_d$  not greater than 0.2 may just follow the rules applying to DCL columns.
- (3) For DCH: In the two lower storeys of the building, the requirements on  $d_{bw}, s_w$  apply over a distance from the end section not less than 1.5 times the critical region length.
- (4) Index  $c$  denotes the full concrete section and index  $o$  the confined core to the centreline of the hoops;  $b_o$  is the smaller side of this core.
- (5)  $\omega_{wd}$  is the ratio of the volume of confining hoops to that of the confined core to the centreline of the hoops, times  $f_{yd}/f_{cd}$ .
- (6)  $\alpha$  is the “confinement effectiveness” factor, computed as  $\alpha = \alpha_s \alpha_n$ ; where:  $\alpha_s = (1-s/2b_o)(1-s/2h_o)$  for hoops and  $\alpha_s = (1-s/2b_o)$  for spirals;  $\alpha_n = 1$  for circular hoops and  $\alpha_n = 1 - \{b_o/((n_h-1)h_o) + h_o/((n_b-1)b_o)\}/3$  for rectangular hoops with  $n_b$  legs parallel to the side of the core with length  $b_o$  and  $n_h$  legs parallel to the one with length  $h_o$ .
- (7) For DCH: at column ends protected from plastic hinging through the capacity design check at beam-column joints,  $\mu_\phi^*$  is the value of the curvature ductility factor that corresponds to 2/3 of the basic value,

$q_o$ , of the behaviour factor used in the design; at the ends of columns where plastic hinging is not prevented because of the exemptions listed in Note (10) below,  $\mu_\phi^*$  is taken equal to  $\mu_\phi$  defined in Note (1) of Table 1 (see also Note (9) below);  $\varepsilon_{sy,d} = f_{yd}/E_s$ .

- (8) Note (1) of Table 1 applies.
- (9) For DCH: The requirement applies also in the critical regions at the ends of columns where plastic hinging is not prevented, because of the waivers listed in Note (10) below.
- (10) The capacity design check does not need to be fulfilled at beam-column joints: (a) of the top floor, (b) of the ground storey in two-storey buildings with axial load ratio  $v_d$  not greater than 0.3 in all columns, (c) if shear walls resist at least 50% of the base shear parallel to the plane of the frame (wall buildings or wall-equivalent dual buildings), and (d) in one-out-of-four columns of plane frames with columns of similar size.
- (11) At a member end where the moment capacities around the joint satisfy:  $\sum M_{Rb} < \sum M_{Rc}$ ,  $M_{Rc}$  is replaced by  $M_{Rc}(\sum M_{Rb}/\sum M_{Rc})$ .
- (12)  $z$  is the internal lever arm, taken equal to  $0.9d$  or to the distance between the tension and the compression reinforcement,  $d-d_1$ .
- (13) The axial load,  $N_{Ed}$ , and its normalized value,  $v_d$ , are taken with their most unfavourable value in the seismic design situation for the shear verification (considering both the demand,  $V_{Ed}$ , and the capacity,  $V_{Rd}$ ).
- (14)  $x$  is the compression zone depth at the end section in the ULS of bending with axial load.

Table 3: EC8 rules for the detailing and dimensioning of ductile walls

	DCH	DCM	DCL
Web thickness, $b_{w0} \geq$	max(150mm, $h_{storey}/20$ )		-
critical region length, $h_{cr} \geq$	$\geq \max(l_w, H_w/6)^{(1)}$ $\leq \min(2l_w, h_{storey})$ if wall $\leq 6$ storeys $\leq \min(2l_w, 2h_{storey})$ if wall $> 6$ storeys		-
<i>Boundary elements:</i>			
a) in critical region:			
- length $l_c$ from edge $\geq$	0.15 $l_w$ , 1.5 $b_w$ , length over which $\epsilon_c > 0.0035$		-
- thickness $b_w$ over $l_c \geq$	200mm; $h_{st}/15$ if $l_c \leq \max(2b_w, l_w/5)$ , $h_{st}/10$ if $l_c > \max(2b_w, l_w/5)$		-
- vertical reinforcement:			
$\rho_{min}$ over $A_c = l_c b_w$	0.5%		0.2% <sup>(0)</sup>
$\rho_{max}$ over $A_c$	4% <sup>(0)</sup>		
- confining hoops (w) <sup>(2)</sup> :			
$d_{bw} \geq$	8mm	in the part of the section where	
spacing $s_w \leq^{(3)}$	min(25 $d_{bh}$ , 250mm)	$\rho_L > 2\%$ : as over the rest of the wall (case c, below)	
$\omega_{wd} \geq^{(2)}$	0.12	0.08	-
$\alpha \omega_{wd} \geq^{(3),(4)}$	30 $\mu_q (v_d + \omega_v) \epsilon_{sv,d} b_w / b_0 - 0.035$		-
b) storey above critical region	As in critical region, but $\alpha \omega_{wd}$ & $\omega_{wd}$ : 50% of those required in critical region	as over the rest of the wall (case c, below)	
c) over the rest of the wall height:	In parts of the section where $\rho_L > 2\%$ : - distance of unrestrained bar in compression zone from nearest restrained bar $\leq 150$ mm; - hoops with $d_{bw} \geq \max(6\text{mm}, d_{bL}/4)$ & spacing $s_w \leq \min(12d_{bL}, 0.6b_{w0}, 240\text{mm})^{(0)}$ up to a distance of $4b_w$ above or below floor beams or slabs, or $s_w \leq \min(20d_{bL}, b_{w0}, 400\text{mm})^{(0)}$ beyond that distance		
<i>Web:</i>			
- vertical bars (v):			
$\rho_{v,min}$	wherever $\epsilon_c > 0.2\%$ : 0.5%; elsewhere 0.2%		0.2% <sup>(0)</sup>
$\rho_{v,max}$	4%		
$d_{bv} \geq$	8mm	-	
$d_{bv} \leq$	$b_{w0}/8$	-	
spacing $s_v \leq$	min(25 $d_{bv}$ , 250mm)	min(3 $b_{w0}$ , 400mm)	
- horizontal bars:			
$\rho_{h,min}$	0.2%	max(0.1%, 0.25 $\rho_v$ ) <sup>(0)</sup>	
$d_{bh} \geq$	8mm	-	
$d_{bh} \leq$	$b_{w0}/8$	-	
spacing $s_h \leq$	min(25 $d_{bh}$ , 250mm)	400mm	
axial load ratio $v_d = N_{Ed}/A_c f_{cd}$	$\leq 0.35$	$\leq 0.4$	-
Design moments $M_{Ed}$ :	If $H_w/l_w \geq 2$ , design moments from linear envelope of maximum moments $M_{Ed}$ from analysis for the "seismic design situation", shifted up by the "tension shift" $a_1$		From analysis for "seismic design situation"
<i>Shear design:</i>			
Design shear force $V'_{Ed}$ = shear force $V'_{Ed}$ from the analysis for "seismic design situation", times factor $\epsilon$ :	if $H_w/l_w \leq 2^{(5)}$ : $\epsilon = 1.2 M_{Rdo}/M_{Edo} \leq q$ if $H_w/l_w > 2^{(5),(6)}$ : $\epsilon = \sqrt{\left(1.2 \frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1 \left(q \frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q$	$\epsilon = 1.5$	$\epsilon = 1.0$
Design shear force in walls of dual systems with $H_w/l_w > 2$ , for $z$ between $H_w/3$ and $H_w$ : <sup>(7)</sup>	$V_{Ed}(z) = \left(\frac{0.75z}{H_w} - \frac{1}{4}\right) \epsilon V_{Ed}(0) + \left(1.5 - \frac{1.5z}{H_w}\right) \epsilon V_{Ed}\left(\frac{H_w}{3}\right)$		From analysis for "seismic design situation"
$V_{Rd,max}$ outside critical region	As in EC2: $V_{Rd,max} = 0.3(1 - f_{ck}(\text{MPa})/250)b_{w0}(0.8l_w)f_{cd}\sin 2\delta$ , with $1 \leq \cot \delta \leq 2.5$		



$V_{Rd,max}$ in critical region	40% of EC2 value	As in EC2
$V_{Rd,s}$ outside critical region	As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{vwd}\cot\delta$	with $1\leq\cot\delta\leq 2.5$
$V_{Rd,s}$ in critical region; web reinforcement ratios. $\rho_h, \rho_v$		
(i) if $\alpha_s=M_{Ed}/V_{Ed}l_w\geq 2$ : $\rho_v=\rho_{v,min}, \rho_h$ from $V_{Rd,s}$ :	As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\delta$	with $1\leq\cot\delta\leq 2.5$
(ii) if $\alpha_s<2$ : $\rho_h$ from $V_{Rd,s}$ : <sup>(8)</sup> $\rho_v$ from: <sup>(9)</sup>	$V_{Rd,s}=V_{Rd,c}+b_{wo}\alpha_s(0.75l_w)\rho_h f_{yhd}$ $\rho_v f_{vwd}\geq \rho_h f_{yhd} N_{Ed}/(0.8l_w b_{wo})$	As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\delta$ with $1\leq\cot\delta\leq 2.5$
Resistance to sliding shear: via bars with total area $A_{si}$ at angle $\pm\phi$ to the horizontal <sup>(10)</sup>	$V_{Rd,s}=A_{si}f_{yd}\cos\phi+$ $A_{sv}\min(0.25f_{yd}, 1.3\sqrt{(f_{yd}f_{cd})})+$ $0.3(1-f_{ck}(MPa)/250)b_{wo}x f_{cd}$	
$\rho_{v,min}$ at construction joints <sup>(9),(11)</sup>	$0.0025, \frac{1.3f_{ctd} - \frac{N_{Ed}}{A_c}}{f_{yd} + 1.5\sqrt{f_{cd}f_{yd}}}$	-

(0) Note (0) of Tables 1 and 2 applies.

(1)  $l_w$  is the long side of the rectangular wall section or rectangular part thereof;  $H_w$  is the total height of the wall;  $h_{storey}$  is the storey height.

(2) For DC M: If for the maximum value of axial force in the wall from the analysis for the “seismic design situation” the wall axial load ratio  $v_d=N_{Ed}/A_c f_{cd}$  satisfies  $v_d\leq 0.15$ , the DCL rules may be applied for the confining reinforcement of boundary elements; these DCL rules apply also if this value of the wall axial load ratio is  $v_d\leq 0.2$  but the value of  $q$  used in the design of the building is not greater than 85% of the  $q$ -value allowed when the DC M confining reinforcement is used in boundary elements.

(3) Notes (4), (5), (6) of Table 2 apply for the confined core of boundary elements.

(4)  $\mu_\phi$  is the value of the curvature ductility factor that corresponds to the product of the basic value  $q_0$  of the behaviour factor times the value of the ratio  $M_{Edo}/M_{Rdo}$  at the base of the wall (see Note (5));  $\varepsilon_{sy,d}=f_{yd}/E_s$ ,  $\omega_{vd}$  is the mechanical ratio of the vertical web reinforcement.

(5)  $M_{Edo}$  is the moment at the wall base from the analysis for the “seismic design situation”;  $M_{Rdo}$  is the design value of the flexural capacity at the wall base for the axial force  $N_{Ed}$  from the analysis for the same “seismic design situation”.

(6)  $S_e(T_1)$  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 multiplied by  $\varepsilon$ ;  $S_e(T_c)$  is the spectral acceleration at the corner period  $T_c$  of the elastic spectrum.

(7) A dual structural system is one in which walls resist between 35 and 65% of the seismic base shear in the direction of the wall shear force considered;  $z$  is distance from the base of the wall.

(8) For  $b_w$  and  $d$  in m,  $f_{ck}$  in MPa,  $\rho_L$  denoting the tensile reinforcement ratio,  $N_{Ed}$  in kN,  $V_{Rd,c}$  (in kN) is given by:

$$V_{R,c} = \left\{ \max \left[ 180(100\rho_1)^{1/3}, 35\sqrt{1 + \sqrt{\frac{0.2}{d}}} f_c^{1/6} \right] \left( 1 + \sqrt{\frac{0.2}{d}} \right) f_c^{1/3} + 0.15 \frac{N}{A_c} \right\} b_w d$$

$N_{Ed}$  is positive for compression and its minimum value from the analysis for the “seismic design situation” is used; if the minimum value is negative (tension),  $V_{Rd,c}=0$ .

(9) The minimum value of the axial force from the analysis for the “seismic design situation” is used as  $N_{Ed}$  (positive for compression).

(10)  $A_{sv}$  is the total area of web vertical bars and of any additional vertical bars placed in boundary elements against shear sliding;  $x$  is the depth of the compression zone.

(11)  $f_{ctd}=f_{ctk,0.05}/\gamma_c$  is the design value of the (5%-fractile) tensile strength of concrete.