RCsolver

THEORY MANUAL

Eurocode design software program (Version 2012)

Version 1.1

Issued: 7-Sep-2012

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In collaboration through a research project with Aristotle University of Thessaloniki

Table of Contents

1. Fundamental Requirements

1.1 Behavior requirements and compliance criteria

Behavior requirements and respective seismic actions

The behavior requirements described in EC8 (§2.1) are:

- **No-collapse requirement:** The structure must be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events has lapsed.
- **Damage limitation requirement:** The structure must be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the cost of which would be dispropotionately high in comparison with the costs of the structure itself.

For common structures the seismic action has a probability of exceedance 10% in 50 years, which means a return period of 475 years. The reference seismic action A_{Ek}, is marked with the letter R. Reliability differentiation is implemented by classifying structures into different importance classes. An importance factor γ_I is assigned to each importance class. Wherever feasible this factor should be derived so as to correspond to a higher or lower value of the return period of the seismic event: $A_{Ed} = \gamma_I A_{Ek}$.

For common structures the seismic action for the damage limitation has a probability of exceedance 10% in 10 years, which means a return period of 95 years. The limitation targets to the reduction of the financial impact in case of a less tense earthquake than of the reference one and the continuation of the operation of buildings important for the national security. EC8 (§2.1 and §4.4.3.2) allows the use of a design seismic action A_{Ed}, multiplied with the reduction factor ν according to the importance class.

These behavior requirements are checked with the compliance criteria that are presented next. Although, EC8 includes a third requirement: the avoid of a total collapse under a rare but undefined seismic action, far bigger than the design seismic action (i.e. with a return period of 2000 years). This requirement targets to avoid total losses, not distinguished victims.

Compliance criteria

The "damage limitation requirement" is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the "no-collapse requirement" (EC8 §4.4.3.2).

Compliance criteria in any kind of project in the requirement of avoiding (even local) collapse under the design seismic action is satisfying the deformation limits or other relevant limits. This happens because earthquake is a dynamic action that, so that the construction must stand not only specific forces, but also an amount of seismic energy that is inserted through the ground and to the respective deformations. This is the reason why EC8 allows the development of important non-elastic deformations, as long as they do not endanger the integrity of the building or of a part of it.

The resistance and energy-dissipation capacity to be assigned to the structure are related to the extent to which its non-linear response is to be exploited. In operational terms such balance between resistance and energy-dissipation capacity is characterized by the values of the behavior factor q and the associated ductility classification, which are given in the relevant Parts of EN 1998. As a limiting case, for the design of structures classified as low-dissipative, no account is taken of any hysteretic energy dissipation and the behavior factor may not be taken, in general, as being greater than the value of 1.5 considered to account for overstrengths. For steel or composite steel concrete buildings, this limiting value of the q factor may be taken as being between 1.5 and 2.

The main way to design seismic-resistant buildings according to is based on ductility. Specifically, they are designed to:

- Have capacity equal to the horizontal forces inserted by the earthquake, divided by the behavior factor $q>1.5$ and
- maintain the capacity to transmit the necessary forces and to dissipate energy under cyclic conditions. To this end, the detailing of connections between structural elements and of regions where nonlinear behaviour is foreseeable should receive special care in design.
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- In order to accomplish the first of the requirements above, the member parts (beam and column edges, wall base) that are needed to develop plastic seismic deformations are designed for the limit falure condition, so that they have the resistance according to forces R_d, at least equal to the elastic tense, E_d, that is inserted through the horizontal forces: (EC8 § 4.4.2.2)

$$R_d \geq E_d \tag{1.1}$$

The design capacity value, R_d, on the limit failure state is calculated as in the design of other actions (i.e. in concrete members with the same values of partial factors, γ_c=1.5, γ_s=1.15).

In order to accomplish the second of the requirements above, the areas where plastic hinges are expected to be formed and their details are designed to have a local plasticity index that ensures the value of the building plasticity, μ_δ. Moreover, It shall be verified that both foundation elements and the foundation soil are able to resist the action effects resulting from the superstructure response without substantial permanent deformations. In determining the reactions, due consideration shall be given to the actual resistance that can be developed by the structural element transmitting the actions.

1.2 Ground conditions and seismic action

Ground conditions

Seismic design can be regarded as the balance between the structural resistance capacity during an earthquake and the expected seismic actions. In practice it is not possible to eliminate seismic damages, mainly because of the large direct cost, and not even desired because as such loading conditions may never occur during the lifetime of the structure.

This is the reason why there is quantitative assessment characterization of the expected seismic excitation levels of the soil. This can be done with the determination of parameters such as the velocity and the deflection, but mainly the Peak Ground Acceleration (PGA). In EC8, seismic action depends not only on the seismic action of the spot of the project, but also on the local soil conditions.

Figure 1.1 presents the seismic risk analysis pictograph. The earthquake model defines earthquake scenarios with size M, at a distance R from the spot of interest, and later using an estimation model of the seismic movements to predict the oscillation parameter in interest for the combination M-R. The results in this case are expressed in terms of acceleration response spectrum.

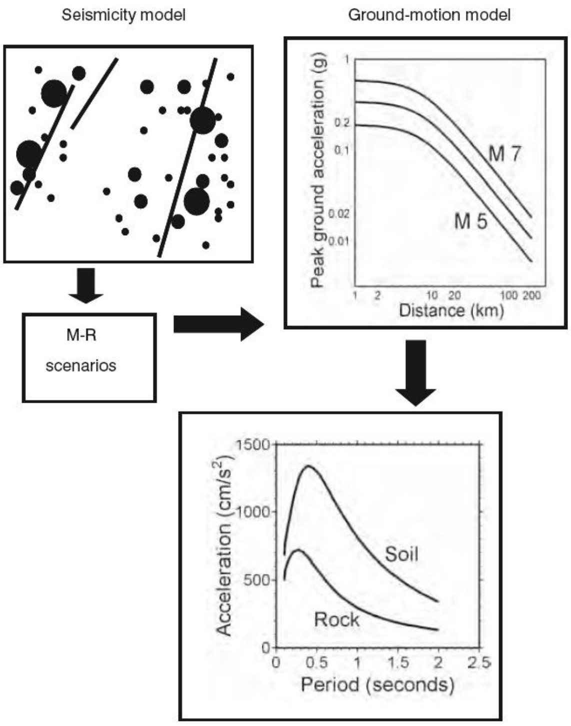


Figure 1.1: The seismic risk analysis pictograph (Elghazouli, 2009)

Seismic action

- Design spectrum in the horizontal direction

The horizontal design forces are defined in EC8 from the maximum response acceleration of the structure, under the expected earthquake, that is represented with the acceleration spectrum of the structure. The starting point is an elastic response spectrum, which is reduced with factors that take into consideration the ability of the structure to absorb seismic energy through rigid deformations. The design acceleration spectrum comes from the elasticity spectrum with a depreciation of 5%, by dividing the spectral accelerations by the behavior factor q. In the horizontal plane, the seismic action acts simultaneously and independently in two orthogonal directions that have the same response spectrum.

EC8 suggests two different design spectrums, Type 1 for the more seismically active regions of southern Europe, and Type 2 for the less seismic regions of central and northern Europe. Spectrum type 1 refers to earthquake sizes close to M7 while spectrum type 2 is suitable for earthquakes up size M5.5.

Figure 1.2 presents average spectral ordinate values from the equations of seismic motion prediction of the European territory by Ambraseys et al. (1996) for rock locations distanced 10 km from small and middle sized earthquakes, in comparison with the spectrum for rock type 1 and type 2 of EC8, based on the average prediction values of the maximum soil acceleration (PGA).

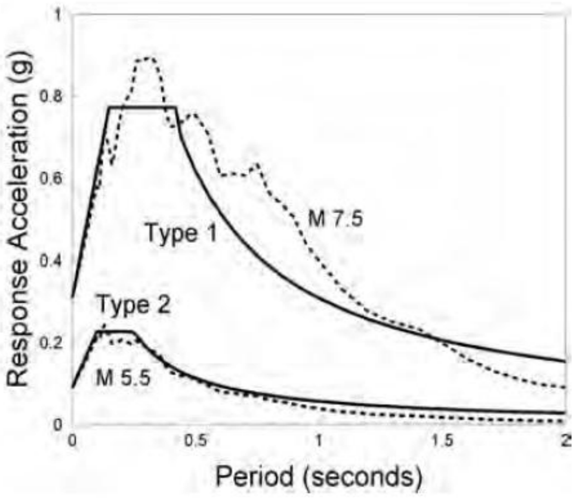


Figure 1.2: Average spectral ordinate values by Ambraseys in comparison with EC8 (Elghazouli, 2009)

The elastic acceleration spectrum with a damping of 5% of EC8 is given graphically below. It contains an area of fixed spectral acceleration, between the periods T_B and T_C with a value 2.5 times the maximum soil acceleration a_gS, that is followed from an area of fixed spectral velocity between the periods T_C and T_D, where the spectral acceleration is proportional to 1/T, and an area of fixed spectral displacement, where the spectral acceleration is proportional to $1/T^2$.

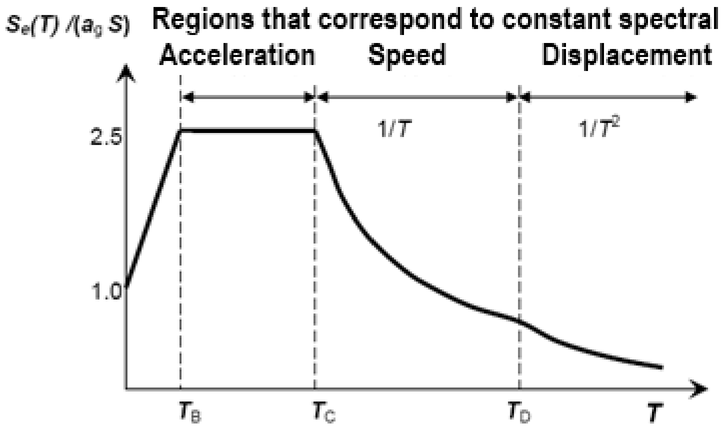


Figure 1.3: Elastic spectrum EC in the horizontal direction for a damping of 5% (Fardis, 2009a)

In the areas of fixed spectral acceleration, velocity, and displacement, the design spectrum originates from an elastic response with a 5% damping divided by q. Exceptionally, the increasing part for a vibration period from T up to $T \leq T_B$ comes from the linear interpolation between: (α) the maximum ground acceleration Sa_g, divided by 1.5, that expresses overstrength compared with the design capacity and the fixed design acceleration, for T=0 and (β) 2.5 a_g/q for $T=T_B$. Moreover, there is a lower limit in the design spectral acceleration, equal to the 20% of the maximum acceleration on the rock, a_g. (Fardis, 2009a)

The following Figure presents the design spectrum for the elastic response spectrum divided by the behavior factor q according to EN 1998 against the ENV 1998 recommendations for q=4.

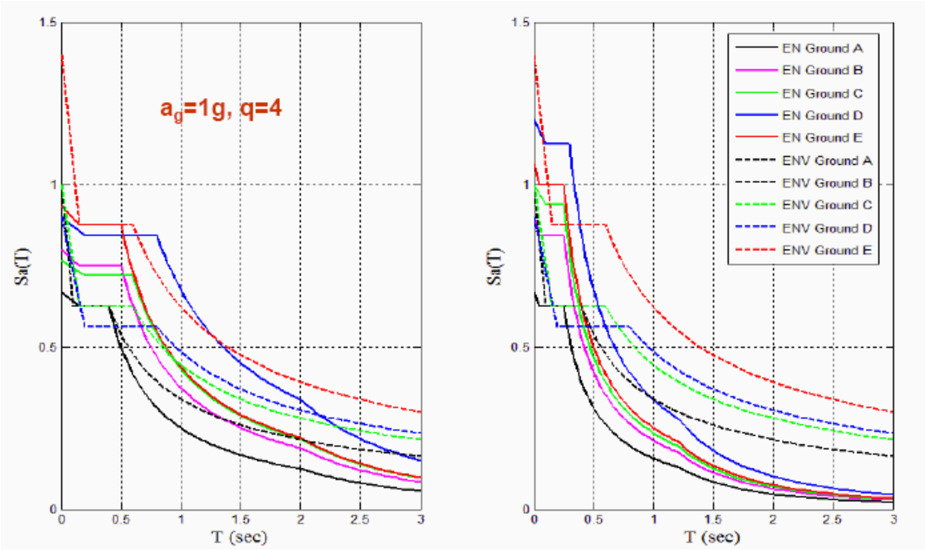


Figure 1.4: Design spectrum (Fardis, 2009c)

The following equations describe the design acceleration spectrum (EC8 § 3.2.2.5(4)) :

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3} \right) \right] \tag{1.2}$$

$$T_B \le T \le T_C : S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \tag{1.3}$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C}{T} \\ \ge \beta \cdot a_g \end{cases} \tag{1.4}$$

$$T_D \le T : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C \cdot T_D}{T^2} \\ \ge \beta \cdot a_g \end{cases} \tag{1.5}$$

where:

S_d (T) is the design spectrum

T is the vibration period of a linear single-degree-of-freedom system

a_g is the design ground acceleration on type A ground (a_g=$\gamma_I \cdot a_{gR}$)

γ_I is the importance factor of the building

T_B is the lower limit of the period of the constant spectral acceleration branch

T_C is the upper limit of the period of the constant spectral acceleration branch

T_D is the value defining the beginning of the constant displacement response range of the spectrum

S is the soil factor

η is the damping correction factor with a reference value of η = 1 for 5% viscous damping

q is the behaviour factor

β is the lower limit of the seismic acceleration (suggested value β=0.2)

- Design spectrum in the vertical direction

The importance of the vertical seismic component in designing structures is open to discussion, but there are certain types of structures or structural members like cantilever beams, for which the vertical action could be critical. Many earthquake standards do not have reference to the vertical elastic or design spectrum and the ones that have, they present it as the horizontal spectrum multiplied by a reduction factor (usually 1/3). Measures of seismic accelerations near the fault have shown that short term the vertical component of the seismic action can be greater than the horizontal one. In addition, it is generally acceptable that frequency content of the vertical response spectrum is different than the horizontal one (Bozorgnia and Campbell, 2004). So, C8 (§3.2.2.3) has the advantage that it defines the vertical response spectrum independently and not depending on the horizontal spectrum. Figure 1.5 presents the ratio between the vertical and horizontal components of the seismic action for soft soils and for Type I spectrum of EC8 in comparison with the average ratios that have been presented by Bozorgnia and Campbell (2004) for soft soils in various distances from the seismic fault.

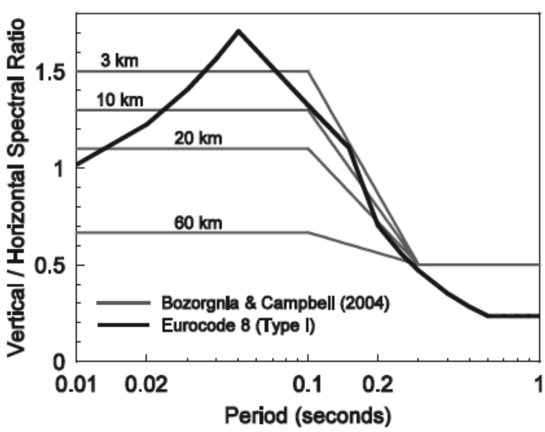


Figure 1.5: The ratio between the vertical and horizontal components of the seismic action for soft soils and for Type I spectrum of EC8 in comparison with actual measures.

The design spectrum of the vertical seismic component is given from the same equations that define the horizontal component, with the difference that the ground factor S is considered equal to 1.0, the behavior factor q is allowed to be considered greater than 1.5 (except from the case that it is documented by appropriate study), the values of the periods T_B, T_C, T_D are different (they are presented in the Table below), the maximum vertical acceleration a_{vg} replaces the $a_{g,}$ in the way that is presented in the same Table, and all the other parameters are received as presented in the previous paragraph.

Spectrum Type	a_{vg}/ a_g	T_B	T_C	T_D
I	0.90	0.05	0.15	1.0
II	0.45	0.05	0.15	1.0

Table 1.1: Parameters of the vertical elastic response spectrum according to EC8 (EC8 § 3.2.2.3 Table 3.4)

1.3 Parameters of the design spectrum

Seismic acceleration factor (a_g)

According to EC8, the dependence of the seismic reference action (i.e. with the possibility of exceeding 10% in 50 years), A_{Ek}, from the geographical location is given on terms of the maximum horizontal reference acceleration a_{gR} on the rock (soil type A) from the national map of Seismic Hazard Zones. For structures with an importance factor different than the usual one (i.e. type II), the maximum design seismic acceleration, a_g, equals to the reference value, a_{gR}, multiplied by the importance factor, $a_g = \gamma_I a_{gR}$.

Importance classes for buildings (γ_I)

Buildings are classified in four importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. The four importance classes are characterized by different importance factors, γ_I, and by reduction factors, v, that are used for determining the design seismic action for the reduction of damages. According to EC8, the importance factor for the class II should be 1.0, and for the rest of the classes it should be defined in the National Annexes. Table 1.2 presents the importance classes as described in EC8.

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Table 1.2: Importance classes for buildings according to EC8
(EC8 § 4.2.5 Table 4.3)

Ground types

According to EC8 (§3.1.2), there are five typical ground types (A, B, C, D, E) and 2 special ground types (S_1, S_2) that may be used to account for the influence of local ground conditions on the seismic action. The average shear wave velocity in the top 30 m from the surface is computed according to the following equation: (EC8 § 3.1.2 (3))

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \quad (1.6)$$

where

h_i and v_i denote the thickness (in meters) and the shear wave velocity (at a shear strain level of 10^{-5} or less) for the i-th formation or layer, in a total of N.

If the value of $v_{s,30}$ is not available, the number of block outs per 0.3 m in N_{SPT} test can be used. If this number is not available either, the undrained cohesion c_u can be used. The following table presents the description of each ground type, and the definition parameters.

Table 1.3: Ground types according to EC8 (EC8 § 3.1.2 Table 3.1)

Ground type and description	$v_{s,30}$	N_{SPT}	c_u
A: Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	-	-
B: Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C: Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D: Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E: A soil profile consisting of a surface alluvium layer with *v*s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with *v*s > 800 m/s.			
S_1: Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	<100	-	10-20
S_2: Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

The following figures present the elastic response spectrums defined by EC8 for each ground type. As mentioned, EC8 (§3.2.2.2(2)) defines 2 spectrum types: Type 1 for regions with high seismic activity (defined as M > 5,5), and Type 2 for regions with average seismic activity (M < 5,5). Spectrums for each ground type are presented that include ground types: A - rock , B – very dense sand, gravel or very stiff clay, C – dense or medium dense sand, gravel or stiff clay, D – loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil, E – soil profiles consisting of a surface alluvium layer with *v*s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material. The vertical axis is the spectral acceleration of an elastic structure normalized to the a_g.

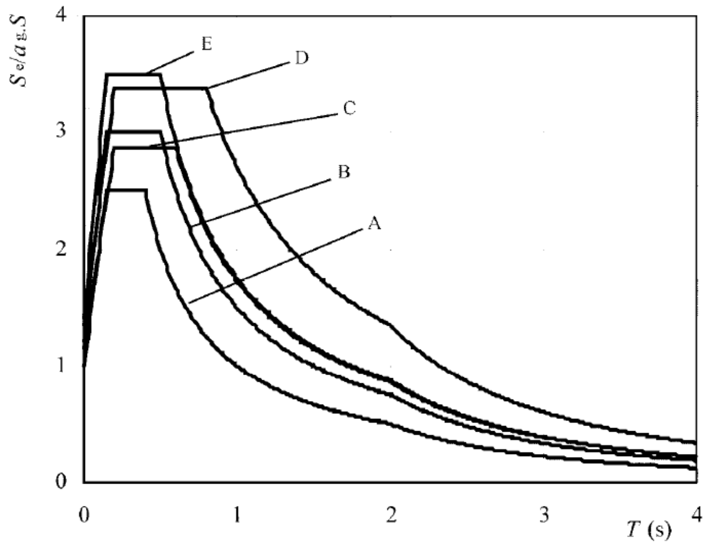


Figure 1.6: Elastic response spectrum Type 1 according to EC8 for damping 5% (EC8 § 3.2.2.2)

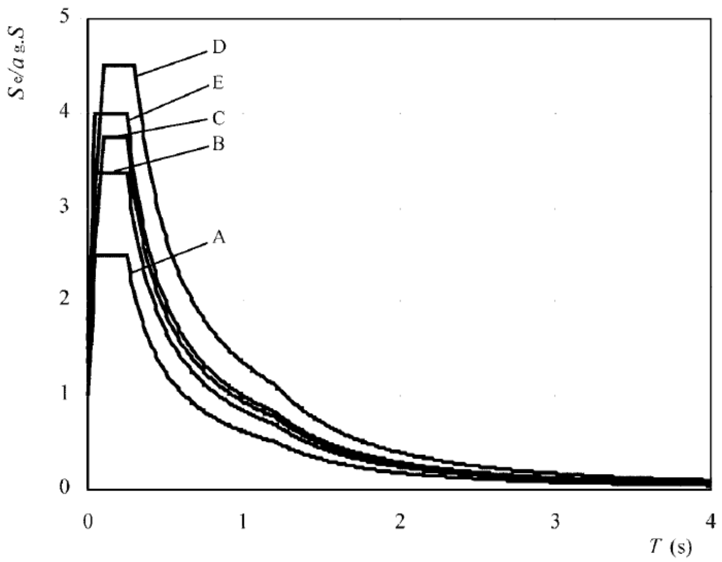


Figure 1.7: Elastic response spectrum Type 2 according to EC8 for damping 5% (EC8 § 3.2.2.2)

Next, Tables 1.4 and 1.5 present the effect of the local ground factors on the seismic actions because of the spectral ground acceleration. The factors that are depending on the ground type are the ground, S, and the periods T_B, T_C and T_D, of which the definition is given right under the following tables. EC8 predicts different values for these factors depending on the elastic response spectrum that has been adopted in each region (Type 1 and Type 2).

Table 1.4: Values for the elastic response spectrum Type 1 according to EC8 (EC8§ 3.2.2.2 Table 3.2)

Ground type	S	T_B	T_C	T_D
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

Table 1.52: Values for the elastic response spectrum Type 2 according to EC8 (EC8 § 3.2.2.2 Table 3.3)

Ground type	S	T_B	T_C	T_D
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

where:

T_B is the lower limit of the period of the constant spectral acceleration branch

T_C is the upper limit of the period of the constant spectral acceleration branch

T_D is the value defining the beginning of the constant displacement response range of the spectrum

S is the soil factor

Damping correction factor (η)

The damping correction factor, η, expresses the variation of the influence of viscosity depreciation in the elastic area of behavior, when the percentage of the critical damping ξ is different than 5%. The increase of the damping in the plastic area of behavior is taken into account in the behavior q. The damping correction factor takes a reference value of η=1 for a rate of viscous damping of the structure equal ξ=5%. The value of the damping correction factor η can be calculated according to the following equation: (EC8 § 3.2.2.2(3))

$$\eta = \sqrt{10/_{5+\xi}} \geq 0.55 \tag{1.7}$$

where

ξ is the viscous damping ratio of the structure, expressed as a percentage.

Behavior factor (q)

The behavior factor depends on the structural material, on the structural model and on other parameters that are being specified here, making obligatory to the designer to estimate an initial value for the behavior factor.

The behavior factor q expresses the ability of a structural model to absorb energy through the inelastic behavior of structural members, without drastic reduction in strength on local and global level. In this way, the seismic accelerations of the structures are reduced, compared to the accelerations that would be applied on a perfectly elastic system. In other words, the behavior factor is an approach of the ratio between the seismic forces that would be applied on the structure if the response would be perfectly elastic with a viscous damping ratio of 5%, and the design seismic actions of a conventional elastic model, that ensures adequate response of the structure.

The value of the behavior factor q, depends on:

- The ductility class
- The type of the structural model
- The regularity of the structure.

Ductility class

EC8 aims to ensure the protection of life during a major earthquake simultaneously with the restriction of damages during more frequent earthquakes. The standard allows the receipt of seismic forces either with damping energy (ductile behavior) or without damping energy (elastic behavior). Nevertheless it is distinguished a preference towards the first approach.

Ductility is defined as the ability of the structure or parts of it to sustain large deformations beyond the yield point without breaking. In the field of applied seismic engineering, the ductility is expressed in terms of demand and availability. The ductility demand is the maximum ductility level that the structure can reach during a seismic action, that is a function of both the structure and the earthquake. The available ductility is the maximum ductility that the structure can sustain without damage and it is an ability of the structure. So, a great part of the standard aims to ensure the existence of a stable and trustworthy model of absorbing energy in predefined critical areas that restrict no inertial loading that appears in other parts of the structure. The designing rules achieve to develop the wanted ductility in these critical areas, with the benefits of the reduced no inertial loading, that are received by more strict construction arrangements and designing rules (Elghazouli, 2009). In the case of reinforced concrete structures, this behavior can be achieved only through the reduction of capacity through delay circles from suitable construction arrangements of such critical zones to ensure stable plastic behavior that it is not undermined by brittle modes of failure such as concrete shearing, concrete crushing, or reinforcement bending.

This leads to the adaptation of three levels of absorbing energy:

- Low (Ductility class low (DCL)) that does not require delayed ductility and the resistanse to seismic loading is achieved through the capacity of the structure (q=1.5).
- Medium (DCM) that allowes high levels of ductility and there are responsive design demands (1.5<q<4).
- High (DCH) that allowes even higher levels of ductility and there are responsive strict and complicated design demands (q>4).

The Ductility Class Low (DCL) predicts the design of the members with the seismic loading that occurs from the design seismic action (of the 475 years) with a behavior factor of q=1.5 and reinforcement calculations like in the case of usual, non-seismic actions, with some material restrictions (the minimum concrete quality that can be used is C16/20 etc). EC8 suggests that the design with DCL should be limited only in areas with low earthquake activity (i.e. in areas with maximum ground design acceleration less than 0.10g). In areas of medium or high earthquake activity, the buildings designed with DCL are not supposed to be financially efficient. In addition, because of the low ductility, it is likely that they would not have a sufficient security level against an earthquake bigger than the design seismic action.

In the two higher ductility classes (DCM and DCH) the design ensures the existence of a stable and trustworthy model of absorbing energy in predefined critical areas and uses a behavior factor q>1.5. These two ductility classes differ in:

- Geometrical restrictions and materials (steel strain)
- The design loadings
- The rules of capacity design and local ductility

The behavior factor can vary in different horizontal directions of the structure, even if the ductility class is the same in all directions.

These two classes are equivalent regarding the performance of the structure under the design seismic action. The design with DCM is easier to be executed on spot and can have a better result in medium seismic actions. The design with DCH seems to provide higher security levels from DCM against local or total collapse under earthquakes greater than the design seismic action. EC8 does not connect the choice of the two higher ductility classes with the seismic actions of the area or the importance of the structure, nor sets any kind of limit for the use of it. It is up to the state-members to define the different areas and the different structure types, or even better to leave the option of choice to the designer.

If the design forces are calculated according to ductile response demand, then it is necessary to ensure that the structure will fail in a ductile way. This demand is the main idea of the capacity design.

The capacity design contents:

- Ensurance of formation of plastic hinges on the beams and not on the columns.
- Providing of sufficient shear reinforcement (dense steel stirrups).
- Ensurance that the steel objects fail far away from the connection points.
- Avoidence of big structural irregularities.
- Ensurance that the tensile capacity will exceed the shear capacity.

The easiest way to define the ductility is in terms of displacements, as the maximum displacement divided with the displacement during the first yield:

$$\mu = \frac{x_{\max}}{x_y} \qquad (1.8)$$

The yield of the structure has as a result the reduction of the maximum load that can be sustained. Usually (as well in EC8) this reduction is being applied through the behavior factor, q:

$$q = \frac{F_{el}}{F_y} \qquad (1.9)$$

where

F_{el} is the maximum strength that would be developed if the system had an elastic response to the seismic action, and F_y is the yield strength of the system.

It is acceptable that, for big periods (>T_C, where T_C is the upper limit of the period of the constant spectral acceleration branch), yielding and elastic structures are subjected to almost the same displacements. Next, for these structures, the force reduction is equal to the ductility. For lower periods, the force reduction that is achieved for a certain ductility is reduced. EC8 uses the following equations: (EC8 § 5.2.3.4(3))

$$\mu = q \quad \text{for} \quad T \geq T_C \qquad (1.10)$$

$$\mu = 1 + (q-1)\frac{T_C}{T} \quad \text{for} \quad T < T_C \qquad (1.11)$$

The first of these equations expresses the rule of equal displacements.(Elghazouli, 2009).

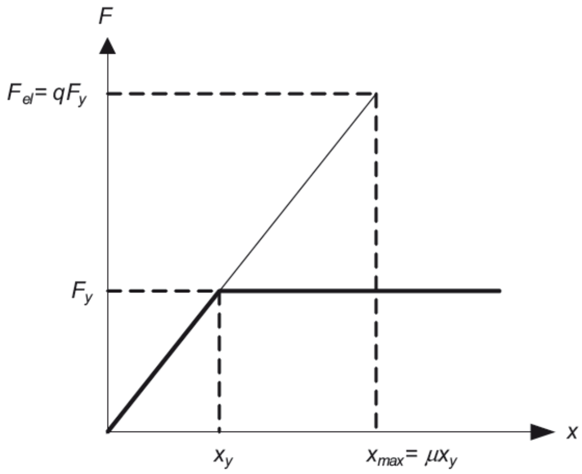


Figure 1.8: Equivalence of ductility and behavior factor, assuming equal elastic and inelastic displacements (Elghazouli, 2009).

Structural types

According to EC8 (§5.2.2.1) concrete buildings should be classified to the following structural types:

- Wall system: structural system in which both vertical and lateral loads are mainly resisted by vertical structural walls, either coupled or uncoupled, whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system
- Frame system: structural system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system
- Dual system: structural system in which support for the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed to in part by the frame system and in part by structural walls, coupled or uncoupled Dual system can be:

- a) Wall equivalent dual system .
- b) Frame equivalent dual system.
- Torsionally flexible system: dual or wall system not having a minimum torsional rigidity
- Inverted pendulum system: system in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building element EC8 does not consider as inverted pendulums one-storey frame systems with beams on both directions, if the axial force $v_d = N_d/A_c f_{cd}$ is less than 0.3 on all columns.
- Ductile wall system. Ductile wall is fixed at its base so that the relative rotation of this base with respect to the rest of the structural system is prevented, and that is designed and detailed to dissipate energy in a flexural plastic hinge zone free of openings or large perforations, just above its base
- System of large lightly reinforced walls. A lightly reinforced wall is a wall with large cross-sectional dimensions, that is, a horizontal dimension *l*w at least equal to 4,0 m or two-thirds of the height *h*w of the wall, whichever is less, which is

expected to develop limited cracking and inelastic behaviour under the seismic design situation

The most recent standards for the design of concrete buildings adopt lower behavior factors for the wall systems in comparison with the frame ones. This happens because:

- Long walls l_w have in general lower percentage of longitudinal reinforcement rather than the beams and the columns that consist frames.
- The actual behavior of walls and wall systems under cyclic loading is less examined than the one of frame systems, because scientific research in walls is more difficult and analytical. As a result, the standards establish larger security margins (Fardis, 2009 b).

Structural regularity

Structures with irregularities in plan or in elevation present special ductility demands in certain locations contrary to the general demand of uniform ductility distribution in normal buildings.

Criteria for regularity

- Criteria for regularity in elevation

EC8(§4.2.3.3) considers as regular in elevation the buildings that satisfy all the following conditions:

- All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.
- Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.
- In framed buildings the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys.
- for gradual setbacks preserving axial symmetry, the setback at any floor shall be not greater than 20 % of the previous plan dimension in the direction of the setback.
- for a single setback within the lower 15 % of the total height of the main structural system, the setback shall be not greater than 50 % of the previous plan dimension (see Figure 1.9.c). In this case the structure of the base zone within the vertically projected perimeter of the upper storeys should be designed to resist at least 75% of the horizontal shear forces that would develop in that zone in a similar building without the base enlargement;
- if the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys shall be not greater than 30 % of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10 % of the previous plan dimension (see Figure 1.9.d).

The irregularity in elevation is expected to have more severe effects in the design and in the seismic response of the building rather than the irregularity in plan. So:

- the static analysis with (equivalent) horizontal seismic loads is allowed to be applied only in buildings that are regular in elevation, and their basic period satisfies for the two main directions the equations: $T \leq 2$ sec and $T \leq 4T_C$.
- In buildings irregular in elevation, the behavior factor q is reduced 20% o in comparison with buildings that are regular in elevation.

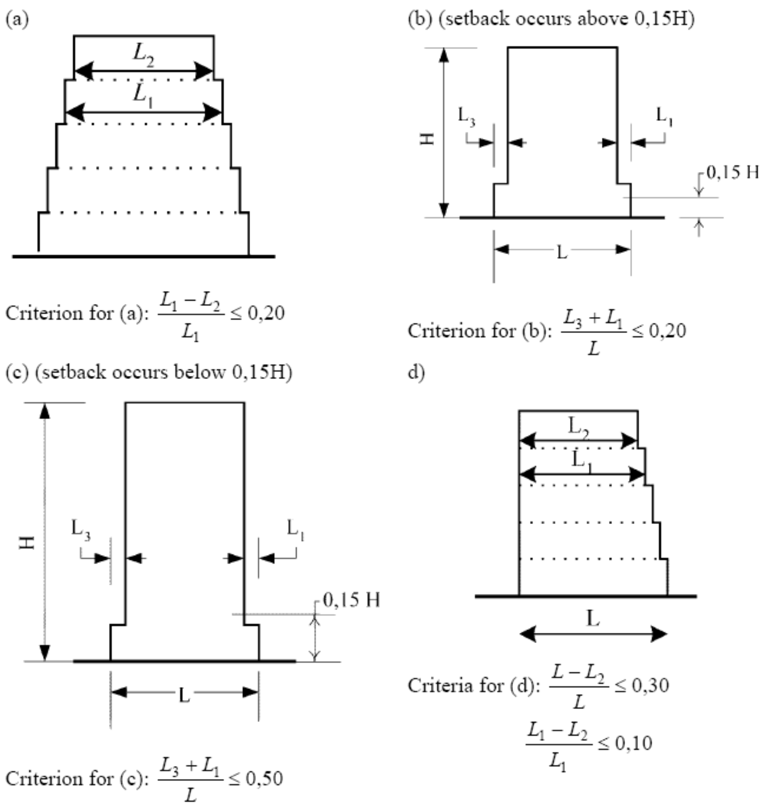


Figure 1.9: Criteria of regularity in elevation for buildings with setbacks (EC8 §4.2.3.3)

Criteria of regularity in plan

EC8(§4.2.3.2) considers as regular in plan the buildings that satisfy all the following conditions:

- With respect to the lateral stiffness and mass distribution, the building structure shall be approximately symmetrical in plan with respect to two orthogonal axes
- The plan configuration shall be compact, i.e., each floor shall be delimited by a polygonal convex line. If in plan set-backs (re-entrant corners or edge recesses) exist, regularity in plan may still be considered as being satisfied, provided that these setbacks do not affect the floor in-plan stiffness and that, for each set-back, the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 5 % of the floor area.
- The in-plan stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements.
- At each level and for each direction of analysis *x* and *y*, the structural eccentricity *e*o and the torsional radius *r* shall be in accordance with the two conditions below, which are expressed for the direction of analysis *y* (EC8 § 4.2.3.2(6)):

$0.3r_x \ge e_x$ and $0.3r_y \ge e_y$ (1.12)

$r_x \ge l_s$ and $r_y \ge l_s$ (1.13)

where the radius of gyration of one floor in the two directions can be calculated from the polar moment of inertia of the floor mass in plan as:

$$r_x = \sqrt{\frac{\Sigma(x^2 EI_y + y^2 EI_x)}{\Sigma(EI_y)}}, \quad r_y = \sqrt{\frac{\Sigma(x^2 EI_y + y^2 EI_x)}{\Sigma(EI_x)}} \tag{1.14}$$

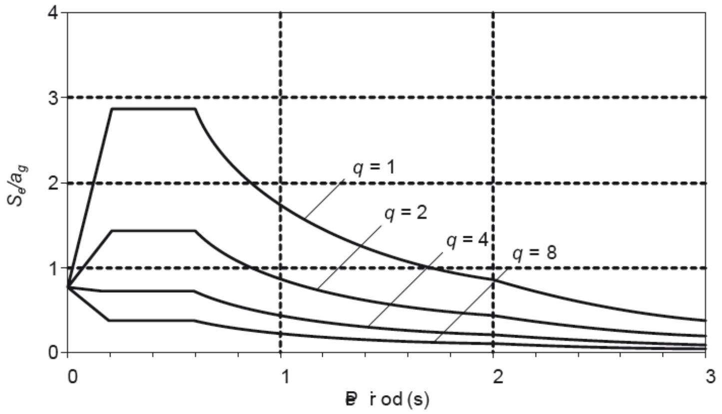


Figure 1.6: Design response spectrum according to EC8 (Spectrum Type 1, soft soils Type C, Elghazouli, 2009).

Provisions of EC8 regarding the value of the behavior factor q

The value of the behavior factor q can be calculated for each direction from the equation: (EC8 §5.2.2.2(1))

$$q = q_0 k_w \geq 1.5 \tag{1.15}$$

where

q_0 is the basic value of the behaviour factor, dependent on the type of the structural system and on its regularity in elevation
k_w is the factor reflecting the prevailing failure mode in structural systems with walls

The following Table presents the basic value of the behavior factor q_0 for systems regular in elevation:

Table 1.7: basic value of the behavior factor q_0 for systems regular in elevation according to EC8 (§ 5.2.2.2 Table 5.1 and Fardis, 2009)).

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3{,}0\alpha_u/\alpha_1$	$4{,}5\alpha_u/\alpha_1$
Uncoupled wall system	3,0	$4{,}0\alpha_u/\alpha_1$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

As mentioned before, for buildings that are not regular in elevation, the basic value of the behavior factor q_0 must be reduced by 20%. The reduction because of irregularity in plan is not obligatory.

The α_u and α_1 are defined as follows:

α_1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant.

α_u is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant.

When the multiplication factor α_u/α_1 has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate values may be used (EC8 §5.2.2.2(5)) :

Frames or frame-equivalent dual systems:

- One-storey buildings: $\alpha_u/\alpha_1 = 1.1$
- multistorey, one-bay frames: $\alpha_u/\alpha_1 = 1.2$
- multistorey, multi-bay frames or frame equivalent structures: $\alpha_u/\alpha_1 = 1.3$

Wall or wall-equivalent dual systems:

- wall systems with only two uncoupled walls per horizontal direction: $\alpha_u/\alpha_1 = 1.0$
- other uncoupled wall systems: $\alpha_u/\alpha_1 = 1.1$
- wall-equivalent dual or coupled wall systems: $\alpha_u/\alpha_1 = 1.2$

For buildings that are not regular in plan, the approximate value of α_u/α_1 is equal to the average of (a) 1.0 and of (b) the value given above. This reduction is not obligatory. When the multiplication factor α_u/α_1 has not been evaluated through an explicit calculation, then the calculated value can be used. In any case, the value of the ratio α_u/α_1 cannot be greater than 1.5. The factor k_w, reflecting the prevailing failure mode in structural systems can be taken as follows (EC8 §5.2.2.2(11)):

- for frames and frame-equivalent systems: $k_w=1.0$
- for walls, wall-equivalent and tortionally flexible systems:

$$1.0 \geq k_w=(1+\alpha_0)/3 \geq 0.5 \qquad (1.16)$$

where

α_0 is the prevailing aspect ratio of the walls of the structural system. If the aspect ratios h_{wi}/l_{wi} of all walls of a structural system do not significantly differ, α_0 can be calculated from the following equation (EC8 §5.2.2.2(12)) :

$$\alpha_0 = \Sigma\, h_{wi}/\Sigma l_{wi} \qquad (1.17)$$

where

h_{wi} is the height of wall I and

l_{wi} is the length of the section of wall i.

Table 1.8 (Fardis, 2009), contains all calculated values of the behavior factor q for different structural systems, depending on the selected ductility class and the regularity of the structure in plan or in elevation.

Structural System	*Regular in plan and in elevation*		*Regular in elevation, Not in plan*		*Regular in plan, Not in elevation*		*Irregular in plan and in elevation*	
	EC8 DCM	DCH	EC8 DCM	DCH	EC8 DCM	DCH	EC8 DCM	DCH
Tortionally flexible	2.0	3.0	2.0	3.0	1.6	2.4	1.6	2.4
Inverted pendulum	1.5	2.0	1.5	2.0	1.5	1.6	1.5	1.6
Wall, with >2 uncoupled walls/direction	3.0	4.4	3.0	4.2	2.4	3.5	2.4	3.35
Wall, with 2 uncoupled walls/direction	3.0	4.0	3.0	4.0	2.4	3.2	2.4	3.2
Mixed wall, wall with all walls coupled or multistorey frame or mixed frame with one span	3.6	5.4	3.3	4.95	2.9	4.3	2.65	3.95
Multistorey frame or mixed frame	3.9	5.85	3.45	5.2	3.1	4.7	2.75	4.15
One-storey frame or mixed frame	3.3	4.95	3.15	4.7	2.65	3.95	2.5	3.8

Table 1.8: Values of the behavior factor of reinforced concrete buildings according to EC8 (Fardis, 2009).

Correlation of the behavior factor member reinforcement details through the curvature ductility factor

The ductility classes medium and high (DCM and DCH) aim, through the seismic design, to control the post-elastic seismic behavior of the structure through the creation of a rigid and strong spine of vertical members, so that the inelastic deformations should be concentrated to the beam edges and to the base of the vertical members, and the formation of the areas of plastic joints so that they can develop plastic Tortional angles, compatible with the behavior factor q that is used in the design. In particular, these areas are formatted in order to have a curvature ductility factor equal to the displacement ductility factor of the building, μ_δ (Fardis, 2009a).

Moreover, EC8 (§5.2.3.4(3)) adopts the following equation between the curvature ductility factor, μ_Φ on the edge of a member and the displacement ductility factor of this member, μ_δ:

$$\mu_\Phi=2\mu_\delta-1 \tag{1.18}$$

It can be easily proved that:

$$\mu_\Phi=2q_0-1 \quad \text{if } T_1\geq T_C \tag{1.19}$$

$$\mu_\Phi=1+2(q_0-1)\, T_C/T_1 \quad \text{if } T_1\leq T_C \tag{1.20}$$

According to the previous value of the curvature ductility factor, μ_Φ as it is chosen from the engineer and the two previous equations, we can calculate:

- the maximum percentage of the tensile reinforcement on the flanges of the beams (EC8 §5.4.3.1.2(4)):

$$\rho_{\max}=\rho'+\frac{0.0018}{\mu_\varphi \varepsilon_{sy,d}}\cdot\frac{f_{cd}}{f_{yd}} \tag{1.21}$$

  where

  $\varepsilon_{sy,d}=f_{yd}/E_s$ is the design value of tension steel strain at yield and the reinforcement ratios of the tension and compression zone, ρ and ρ’. If the tension zone includes a slab, the amount of slab reinforcement parallel to the beam within the effective flange width is included in ρ.

- the mechanical volumetric ratio of confining hoops within the critical regions (i.e. : (a) to the base of columns and walls and (b) to the critical regions on the edges of columns in DCH that do not satisfy the capacity design of the member). To the critical regions on the edges of columns in DCH that the capacity design of the member is satisfied, confining hoops are placed with mechanical volumetric ratio $a\omega_{wd}$ that is calculated according to the reduced value of the curvature ductility factor, that is equivalent to a behavior factor value equal to 2/3 of the basic value q_o.

The mechanical volumetric ratio of confining hoops $a\omega_{wd}$ is given through the following equation: (EC8 §5.4.3.2.2(8))

$$a\omega_{wd}\geq 30\mu_\varphi(\nu_d+\omega_v)\varepsilon_{sy,d}\frac{b_c}{b_o}-0.035 \tag{1.22}$$

where

$\nu_d = N_{Ed} / A_c f_{cd}$ is the normalized design axial force

$\omega_v = \rho_v f_{yd,v} / f_{cd}$ the mechanical volumetric ratio of confining hoops within the web of the wall

b_c is the gross cross-sectional width

b_o is the width of the confined core (to the centerline of the hoops)

α is the confinement effectiveness factor

2. EUROCODE PROVISIONS –NATIONAL ANNEXES

2.1 Design Parameters

Factors of Safety (Eurocode 2-1.1 (§ 2.4.2.4))

For the ultimate limit state, the partial safety factors γ_c και γ_s should be used for the materials. The values of the factors γ_c and γ_s are defined for each country through the National Annexes. The recommended values for persistent, transient and accidental design situations are presented in the Table 2.1.

Design situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent and Transient	1.50	1.15	1.15
Accidental	1.20	1.00	1.00

Table 2.1: Partial factors for materials for ultimate limit states. (Eurocode 2 - Πίνακας 2.1N)

Combination of actions

When more than one transient actions occur are applied concurrently, different combinations of actions should be examined. For this reason, the combination factors ψ are used, that are different for different actions. Eurocode 2 defines three factors ψ_0, ψ_1 and ψ_2 for the combination of actions.

Actions	ψ_0	ψ_1	ψ_2
Loads on buildinfs, type			
Type A: houses, domestic buildings	0.70	0.50	0.30
Type B: offices	0.70	0.50	0.30
Type C: aggregation sites	0.70	0.70	0.60
Type D: comercial sites	0.70	0.70	0.60
Type E: storage facilities	1.00	0.90	0.80
Type F: traffic sites			
vehicle weight ≤ 30 kN	0.70	0.70	0.60
Type G: traffic sites			
30 kN < vehicle weight ≤ 160 kN	0.70	0.50	0.30
Type H: roofs	0	0	0
Snow loadings on buildings (see EN 1991-1-3)			
Finland, Iceland, Norway, Sweeden	0.70	0.50	0.20
Rest CEN state-members for locations			
with elevation H > 1000 m	0.70	0.50	0.20
Rest CEN state-members for locations			
with elevation H ≤ 1000 m	0.50	0.20	0
Wind loads on buildings (see EN 1991-1-4)	0.60	0.20	0
Temperature on buildings (not fire) (see EN 1991-1-5)	0.60	0.50	0

Table 2.2: Recommended values for factor ψ for buildings. (Eurocode 0- Table A1.1)

2.3 Materials

Modulus of elasticity

The modulus of elasticity of a concrete is controlled by the moduli of elasticity of its components. Approximate values for the modulus of elasticity E_{cm}, secant value between $\sigma_c = 0$ and 0,4 f_{cm}, for concretes with quartzite aggregates, are given in Table 2.3. For limestone and sandstone aggregates the value should be reduced by 10% and 30% respectively. For basalt aggregates the value should be increased by 20%. The strength classes in this code are based on the characteristic cylinder strength *f*ck determined at 28 days with a maximum value of *C*max. Eurocode 2-1.1 (§ 3.1.3)

Table 2.3: Strength and deformation characteristics for concrete (Eurocode 2 – Table 3.1)

	Strength classes for concrete													
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90
$f_{ck.cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98
f_{ctm} (MPa)	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	4.8	5.0
$f_{ctk,0,05}$ (MPa)	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9	3.0	3.1	3.2	3.4	3.5
$f_{ctk,0,95}$ (MPa)	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3	5.5	5.7	6.0	6.3	6.6
E_{cm} (Gpa)	27	29	30	31	32	34	35	36	37	38	39	41	42	44
ε_{c1}(‰)	1.8	1.9	2.0	2.1	2.2	2.25	2.3	2.4	2.45	2.5	2.6	2.7	2.8	2.8
ε_{cu1}(‰)	3.5									3.2	3.0	2.8	2.8	2.8
ε_{c2}(‰)	2.0									2.2	2.3	2.4	2.5	2.6
ε_{cu2}(‰)	3.5									3.1	2.9	2.7	2.6	2.6
n	2.0									1.75	1.6	1.45	1.4	1.4
ε_{c3} (‰)	1.75									1.8	1.9	2.0	2.2	2.3
ε_{cu3} (‰)	3.5									3.1	2.9	2.7	2.6	2.6
f_{ctm} (MPa)	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	4.8	5.0
$f_{ctk,0,05}$ (MPa)	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9	3.0	3.1	3.2	3.4	3.5
$f_{ctk,0,95}$ (MPa)	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3	5.5	5.7	6.0	6.3	6.6
E_{cm} (Gpa)	27	29	30	31	32	34	35	36	37	38	39	41	42	44

Design compressive and tensile strengths (Eurocode 2-1.1 (§ 3.1.6))

The value of the design compressive strength is defined as:

$$f_{cd} = a_{cc} \cdot \frac{f_{ck}}{\gamma_c} \qquad (2.1)$$

where:

γ_c is the partial safety factor for concrete

a_{cc} is a coefficient taking into account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied. This value is defined in the National Annex of each country. The recommended value is 1.0.

The value of the design tensile strength is defined as:

$$f_{ctd} = a_{ct} \cdot \frac{f_{ctk,0.05}}{\gamma_c} \qquad (2.2)$$

where :

γ_c is the partial safety factor for concrete

a_{ct} is a coefficient taking into account of long term effects on the tensile strength and of unfavorable effects resulting from the way the load is applied. This value is defined in the National Annex of each country. The recommended value is 1.0.

Stress-strain relations for the design of cross-sections

The designer can use different types of stress-strain diagrams, related with the structure type and the special parameters of the design. For the design of cross-sections, the following stress-strain relationship may be used: Eurocode 2-1.1 (§ 3.1.7)

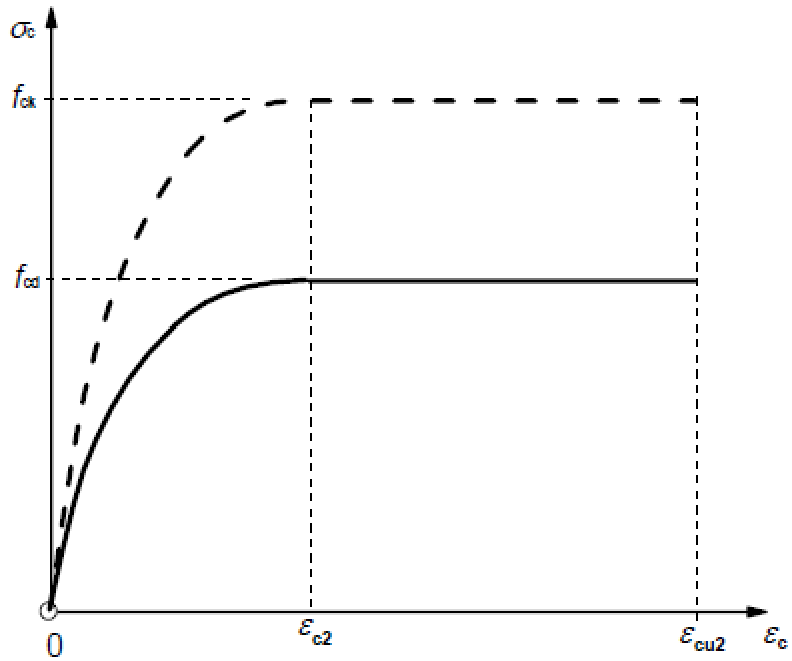


Figure 2.1: Παραβολικό – rectangle diagram of concrete under compression (EC2 - Figure 3.3)

where:

σ_c is the compressive strain

$$\sigma_c = f_{cd} \cdot \left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] \text{ for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \quad (2.3)$$

$$\sigma_c = f_{cd} \quad \text{for } \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2} \quad (2.4)$$

where:

f_{cd} is the design value of concrete compressive strength

n exponent

$$n = 1.4 + 23.4\,[(90 - fck)/100] \quad \text{for } f_{ck} \geq 50\,MPa \text{, else 2.0} \quad (2.5)$$

$$n = 2.0 \quad \text{for } f_{ck} < 50\,MPa \quad (2.6)$$

ε_{c2} is the strain at reaching the maximum strength

$$\varepsilon_{c2}(‰) = 2.0 + 0.085 \cdot (f_{ck} - 50)^{0.53} \quad \text{for } f_{ck} \geq 50\,MPa \quad (2.7)$$

$$\varepsilon_{c2}(‰) = 2.0 \quad \text{for } f_{ck} < 50\,MPa \quad (2.8)$$

ε_{cu2} is the ultimate strain

$$\varepsilon_{cu2}(‰) = 2.6 + 35 \cdot [(90 - f_{ck})/100]^4 \quad \text{for } f_{ck} \geq 50\,MPa \quad (2.9)$$

$$\varepsilon_{cu2}(‰) = 3.5 \quad \text{for } f_{ck} < 50\,MPa \quad (2.10)$$

2.4 Rectangular stress distribution

When the cross-section is not totally under compression, then a simplified rectangular stress distribution can be used.

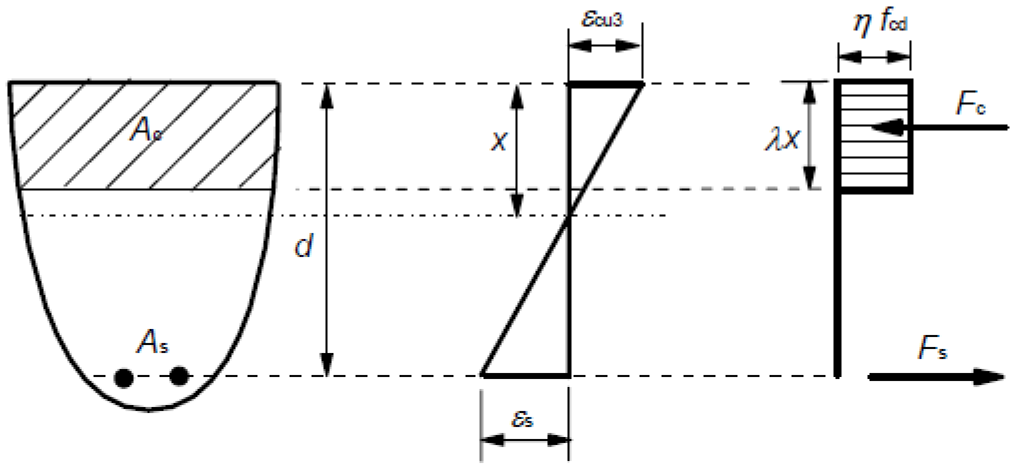


Figure 2.2: Rectangular stress distribution. (Eurocode 2 – Equation 3.5)

According to Eurocode 2-1.1 the factor λ, defining the effective height of the compression zone and the factor η, defining the effective strength, follow from:

$$\lambda = 0.8 \; for \; f_{ck} \leq 50 \; MPa \tag{2.15}$$

$$\lambda = 0.8 - (f_{ck} - 50)/400 \;\text{ for }\; 50 < f_{ck} \leq 90 \; MPa \tag{2.16}$$

and

$$\eta = 1.0 \text{ for } f_{ck} \leq 50 \; MPa \tag{2.17}$$

$$\eta = 1.0 - (f_{ck} - 50)/200 \text{ for } 50 < f_{ck} \leq 90 \; MPa \tag{2.18}$$

Reinforcement steel

According to Eurocode 2-1.1 for normal design we can assume an inclined top branch with a strain limit of ε_{ud} and a maximum stress of kf_{yk}/γ_s at ε_{uk}, where $k = (f_t/f_y)$k, or a horizontal top branch without the need to check the strain limit. Figure 2.6 presents the idealized and design stress-strain diagrams for reinforcing steel for tension and compression.

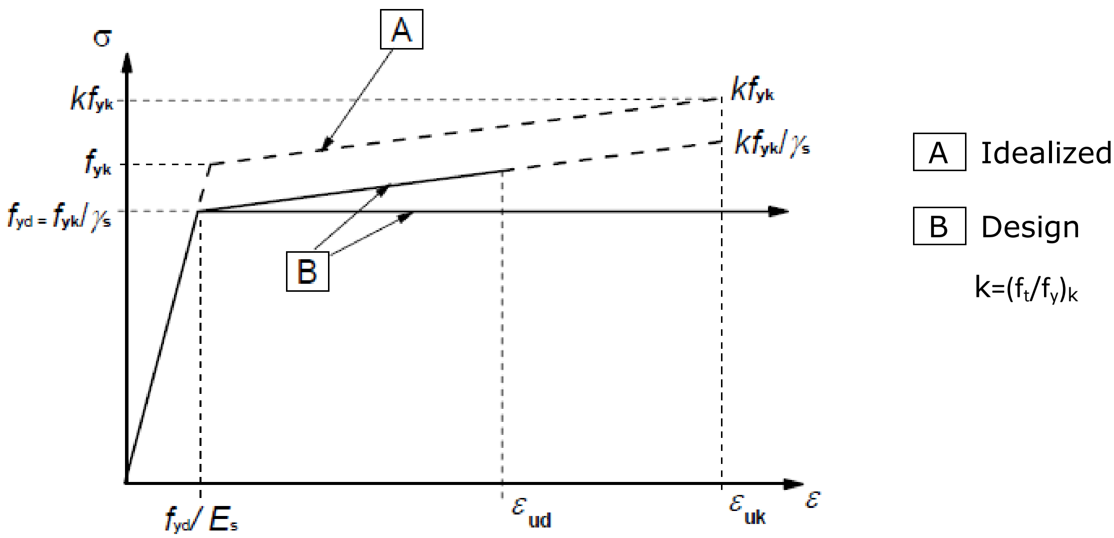


Figure 2.3: Idealized and design stress-strain diagrams for reinforcing steel (tension and compression). (Eurocode 2 – Figure 3.8)

2.5 Durability and cover to reinforcement

Environmental conditions

The required protection of the structure should be established by considering its intended use, design working life, maintenance program and actions. The environmental conditions should be considered during the deign working life of the structure and must be considered during the design, so that they are evaluated in terms of durability. Eurocode 2-1.1 (§4.2) defines the following six environmental conditions:

- XO : No risk of corrosion or attack
- XC1, XC2, XC3, XC4 : Corrosion induced by carbonation
- XD1, XD2, XD3 : Corrosion induced by clorides
- XS1, XS2, XS3 : Corrosion induced by clorides from sea water
- XF1, XF2, XF3, XF4 : Freeze/Thaw attack
- XA1, XA2, XA3 : Chemical attack

Concrete cover

Eurocode 2-1.1 has special requirements for calculating of the minimum concrete cover. The minimum cover, C_{min}, should be provided in order to ensure (a) the safe transmission of bond forces, (b) the protection of the steel against corrosion and (c) an adequate fire resistance. The nominal cover should be specified on the drawings, and it is defined as the minimum cover, plus an allowance in design for deviation, Δ_{cdev}

$$c_{nom} = c_{min} + \Delta_{cdev} \quad (2.19)$$

The greater value of c_{min} satisfying both environmental and bond conditions should be used:

$$c_{min} = max\,\{c_{min,b}\,, c_{min,dur} + \Delta_{cdur,\gamma} - \Delta_{cdur,st} - \Delta_{cdur,add}\,, 10\,mm\} \quad (2.20)$$

where:

$c_{min,b}$ minimum cover due to bond requirement

$c_{min,dur}$ minimum cover due to environmental conditions

$\Delta_{cdur,\gamma}$ additive safety element

$\Delta_{cdur,st}$ reduction of minimum cover for use of stainless steel

$\Delta_{cdur,add}$ reduction of minimum cover for use of additional protection

2.6 Geometric data

Effective width of flanges

In T-section beams the effective width, over which uniform conditions of stress can be assumed, depends on the web and flange dimensions, the type of loading, the span, the support conditions and the transverse reinforcement. The effective flange width b_{eff} of a T-section beam or an L-section beam is derived according to Eurocode 2-1.1 from the equation 2.21 below.

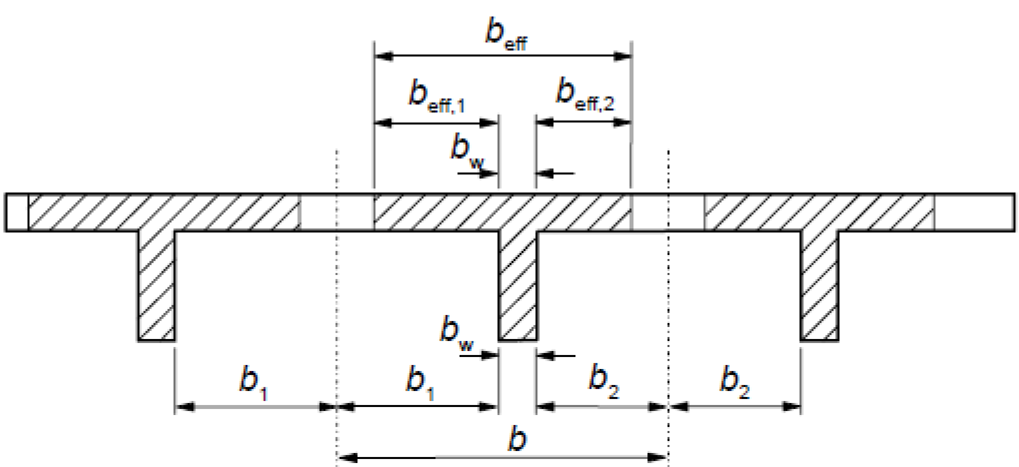


Figure 2.4: Effective flange width parameters (Eurocode 2– Figure 5.3)

$$b_{eff} = \sum b_{eff,i} + b_w \leq b \quad (2.21)$$

$$b_{eff,i} = min\,\{\,0.2 \cdot b_i + 0.1 \cdot l_0\,;\, 0.2 \cdot l_0\,;\, b_i\} \quad (2.22)$$

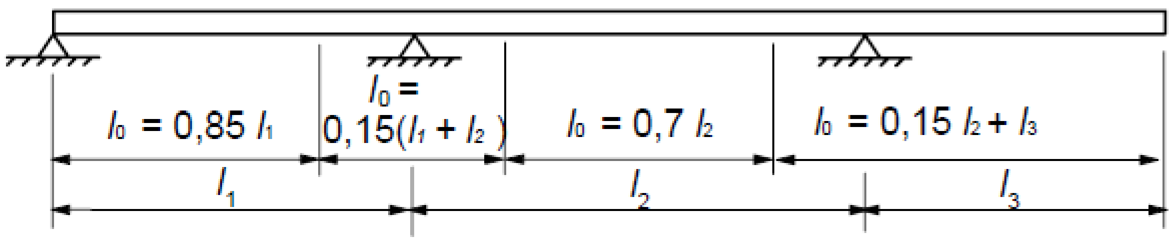


Figure 2.5: Definition of l_0 for calculation of effective flange width (Eurocode 2– Figure 5.2)

2.7 Shear

Design against shear stress according to EN 1992-1-1

According to Eurocode 2-1.1, no calculated shear reinforcement is necessary in regions that the shear force is less or equal to the value of the design shear resistance of the member without shear reinforcement, $V_{Rd,c}$. If $V_{Rd,c} > V_{Ed}$ then minimum shear reinforcement percentage is predicted only for the beams. If the design shear force is greater than the value $V_{Rd,c}$ ($V_{Rd,c} < V_{Ed}$) then sufficient shear reinforcement should by applied. The design of members with shear reinforcement is based on a truss model. The angle θ should be limited according to the limitation: $1 \leq \cot\theta \leq 2.5$ (i.e. $45° \leq \theta \leq 21.8°$).

The next flow chart presents the shear reinforcement design as described in Eurocode 2.

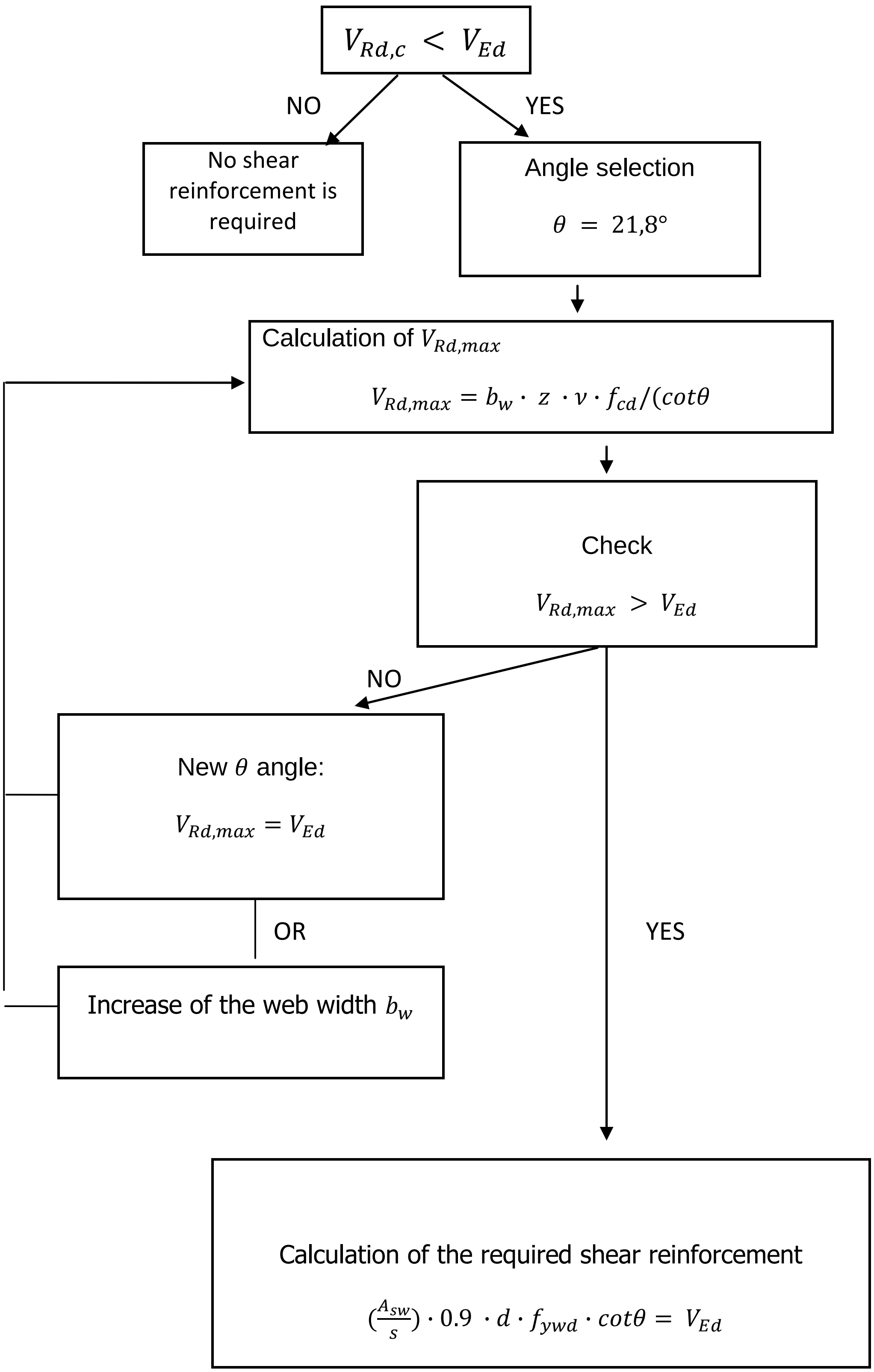


Figure 2.6: Shear reinforcement design procedure according to Eurocode 2-1.1.

Design of members where no shear reinforcement is required

For the shear design of members where no shear reinforcement is required, the design shear capacity $V_{Rd,c}$ can be calculated according to Eurocode 2-1.1 (§6.2.2) from the equation:

$$V_{Rd,c} = [\, C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d \quad (2.23)$$

$$V_{Rd,c} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d \quad (2.24)$$

where:

$C_{Rd,c}$ recommended value $0.18/\gamma_c = 0.12$

$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$

$$\rho_l = \frac{A_{sl}}{b_w \cdot d} \leq 0.02$$

A_{sl} is the area of the tensile reinforcement, which extends $(d + l_{bd})$ beyond the section considered

k_1 recommended value 0.15

f_{ck} is the characteristic compressive cylinder strength of concrete

$\sigma_{cp} = \frac{\mathrm{N_{Ed}}}{\mathrm{A_c}} < 0.2 \cdot \mathrm{f_{cd}}$

$\mathrm{N_{Ed}}$ is the axial force on the cross-section due to loading, measured in N

$\mathrm{A_c}$ is the area of the cross-section measured in mm^2

b_w is the smallest width of the cross-section in the tensile area

d is the effective depth of the cross-section

$$v_{min} = 0.035 \cdot k^{3/2} \cdot f_{ck}{}^{1/2}$$

Design of members requiring shear reinforcement

If shear reinforcement is required ($V_{Rd,c} < V_{Ed}$), the required reinforcement is defined according to a truss model (Figure 2.10). For members with vertical shear reinforcement, Eurocode 2-1.1 (§6.2.3) defines the shear resistance V_{Rd} as the smaller value of the following:

$$V_{Rd,s} = \frac{A_{sw}}{s} z \cdot f_{ywd} \cdot cot\theta \quad (2.25)$$

$$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}/(cot\theta + tan\theta) \quad (2.26)$$

where:

α_{cw} is a coefficient taking account of the state of the stress in the compression chord. (it is defined in each countries National Annex, the recommended value is 1 for not prestressed members)

$v_1 = v$ is a strength reduction factor for concrete cracked in shear

$v = 0.6 \cdot [1 - f_{ck}/250]$ (2.27)

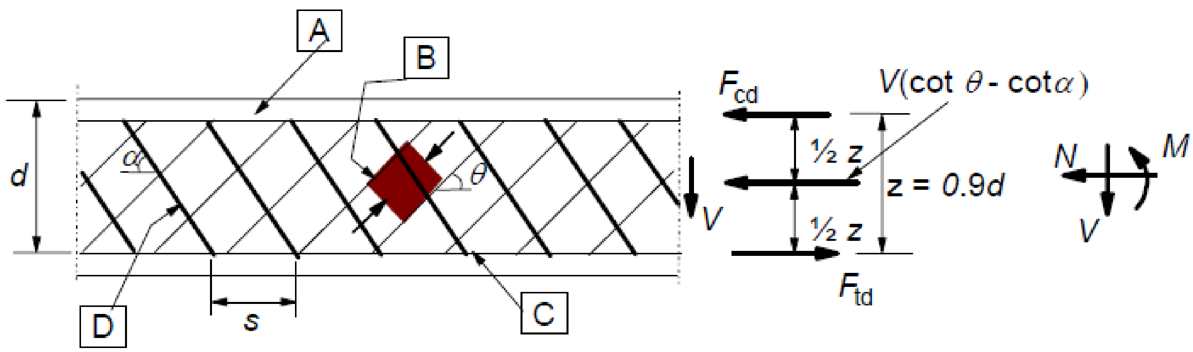
f_{ywd} is the design yield strength of the shear reinforcement

s is the spacing of the stirrups

A_{sw} is the cross-sectional area of the shear reinforcement

$z = 0.9 \cdot d$ is the lever arm of internal forces

θ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force



A compression chord B struts C tension chord

D shear reinforcement

Figure 2.70: Truss model for members with shear reinforcement (EC2– Figure 6.5)

The advantage of the design with the model of the variable angle ϑ is that it provides freedom during the design, as small ϑ angles lead to low reinforcement requirement (A_{sw}), meanwhile large ϑ angles lead to thin webs (reduction of concrete amount. At the same time, it is a simple equilibrium model. Nevertheless, the use of the model of the variable angle presents also disadvantages, like the fact that $\theta_{calculated} \neq \theta_{actual}$, that the model does not totally confront with the trend and that there occur problems during the seismic design. (Penelis G. – Kappos A. - Ignatakis C. - Sextos A., 2009).

2.8 Serviceability limit state

Crack control (Eurocode 2-1.1 (§7.3.1))

Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable. Cracking is normal in reinforced concrete structures subject to bending, shear, torsion or tension resulting from either direct loading or restraint or imposed deformations. Cracks may also arise from other causes such as plastic shrinkage or expansive chemical reactions within the hardened concrete. Such cracks may be unacceptably large but their avoidance and control lie outside the scope of Eurocode 2-1.1.

Cracks may be permitted to form without any attempt to control their width, provided they do not impair the functioning of the structure. A limiting calculated crack width, w_{max}, taking into account the proposed function and nature of the structure and the costs of limiting cracking, should be established. W_{max} is defined in the National Annexes. The recommended values are presented in Table 2.4 below.

Exposure class	Reinforced members and prestressed members with unboned tendons	Prestressed members with boned tendons
	Quasi-permanent load combination	**Frequent load combination**
X0, XC1	0.4[1]	**0.2**
XC2,XC3,XC4	0.3	**0.2[2]**
XD1,XD2,XS1, XS2, XS3		**Decompression**

Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is to set guarantee acceptable appearance. In the absence of appearance conditions, this limit may be relaxed.

Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

Table 2.4: Recommended values of w_{max}(mm). (Eurocode 2– Table 7.1N)

Minimum reinforcement areas

If crack control is required, a minimum amount of bonded reinforcement is required to control cracking in areas where tension is expected. The amount may be estimated from equilibrium between the tensile force in concrete just before cracking and the tensile force in reinforcement at yielding or at a lower stress if necessary to limit the crack width. Eurocode 2-1.1 (§7.3.2) suggests that unless a more rigorous calculation shows lesser areas to be adequate, the required minimum areas of reinforcement may be calculated as follows:

The minimum area of the reinforcing steel in the tensile area can be calculated from the following equation:

$$A_{s,min} \cdot \sigma_s = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} \tag{2.28}$$

where:

$A_{s,min}$ is the minimum area of the reinforcing steel in the tensile area

A_{ct} is the area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack

σ_s is the absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack. This may be taken as the yield strength of the reinforcement, f_{yk}. A lower value may, however, be needed to satisfy the crack width limits according to the maximum bar size or spacing.

$f_{ct,eff}$ η μέση τιμή της εφελκυστικής αντοχής του σκυροδέματος που ισχύει τη στιγμή που τα ρήγματα αναμένεται να δημιουργηθούν για πρώτη φορά: $f_{ct,eff}$= f_{ctm} ή χαμηλότερη, ($f_{ctm}(t)$), αν η ρηγμάτωση αναμένεται να συμβεί πριν τις 28 ημέρες

k is a coefficient which allows the effect of non-uniform self equilibrating stresses, which lead to a reduction of restraint forces.

$k = 1.0$ for webs with $h \leq 300\ mm$ or flanges with widths $< 300\ mm$

$k = 0.65$ for webs with $h \geq 800\ mm$ or flanges with widths $> 800\ mm$, intermediate values may be interpolated

k_c is a coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm:

For pure tension $k_c = 1.0$

For bending or bending combined with axial force:

$$k_c = 0.4 \cdot \left(1 - \frac{\sigma_c}{\left[k_1 \cdot \left(\frac{h}{h^*}\right) \cdot f_{ct,eff}\right]}\right) \leq 1 \qquad (2.29)$$

For rectangular sections and webs of box sections and T-sections

$$k_c = 0.9 \cdot \frac{F_{cr}}{f_{ct,eff} \cdot A_{ct}} \geq 0.5 \qquad (2.30)$$

For flanges of box sections and T-sections

where:

σ_c is the mean stress of the concrete acting on the part of the section under consideration

$$\sigma_c = N_{Ed}/(b \cdot h) \qquad (2.31)$$

N_{Ed} is the axial force at the serviceability limit state

$$\mathrm{h}^* = \begin{cases} \mathrm{h\ for\ h} \leq 1.0\ \mathrm{m} \\ 1.0\ \mathrm{for\ h} \geq 1.0\ \mathrm{m} \end{cases}$$

k_1 is a coefficient considering the effects of axial forces on the stress distribution:

$k_1 = 1.5$ if N_{Ed} is a compressive force

$k_1 = 2 \cdot h^*/(3 \cdot h)$ if N_{Ed} is a tensile force

F_{cr} is the absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment calculated with $f_{ct,eff}$

Control of cracking without direct calculation (Eurocode 2-1.1 (§7.3.3))

For reinforced or prestressed slabs in buildings subjected to bending without significant axial tension, specific measures to control cracking are not necessary where the overall depth does not exceed 200 mm and relative provisions have been applied. When the minimum reinforcement is provided, crack widths are unlikely to be excessive if:

- for cracking caused dominantly by restraint, the bar sizes given in Table 2.6 are not exceeded where the steel stress is the value obtained immediately after cracking,
- for cracks caused mainly by loading, either the provisions of Table 2.5 or the provisions of Table 2.6 below are complied with. The steel stress should be calculated on the basis of a cracked section under the relevant combination of actions.

Steel Stress	Maximum bar spacing [mm]		
[MPa]	w_k=0.4 mm	w_k=0.3 mm	w_k=0.2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Table 2.5: Maximum bar spacing for crack control (Eurocode 2– Table 7.3N)

Steel Stress	Maximum bar size [mm]		
[MPa]	w_k=0.4 mm	w_k=0.3 mm	w_k=0.2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 2.6: Maximum bar diameters ϕ^*_s for crack control (Eurocode 2– Table 7.2N)

2.9 Detailing of reinforcement in concrete members

Maximum and minimum required reinforcement

Here is presented the minimum and maximum reinforcement, as defined by Eurocode 2-1.1 for beams and columns.

- 1. Beams

According to paragraph §9.2.1.1 of Eurocode 2-1.1, the minimum and maximum longitudinal tensile reinforcement that are required in a concrete beam are calculated using the following equations:

$$A_{s,min} = 0.26 \cdot f_{ctm}\, b_t \cdot d / f_{yk} \geq 0.0013 b_t \cdot d \qquad (2.32)$$

where: b_t is the mean width of the tension zone

and

$$A_{s,max} = 0.04 \cdot A_c \text{ (recommended value)} \qquad (2.33)$$

- 2. Columns

According to paragraph §9.5.2 of Eurocode 2-1.1, the minimum and maximum longitudinal tensile reinforcement that are required in a concrete column are calculated using the following equations:

$$A_{s,min} = 0.10 \cdot N_{Ed} / f_{yd} \geq 0{,}002 \cdot A_c \qquad (2.34)$$

$$A_{s,max} = 0.04 \cdot A_c \text{ (recommended value)} \qquad (2.35)$$