European Standards for Reinforced Concrete

– Eurocode 2 for the design for strength, service and durability –

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ABSTRACT: The Structural Eurocodes of the European Communities establish requirements for building and civil engineering works in terms of reliability, adequate performance in service conditions and durability. For the achievement of these requirements, several steps are necessary in the design process. They are subject of Eurocode 2 "Design of Concrete Structures" and the European Standard EN 206 for concrete technology. The basic elements of this integrated design concept are described in the present paper, regarding in particular the requirements for reinforcing steel.

Keywords: European Standards for the Design of Concrete Structures, Design Working Life, Durability, Requirements for Reinforcing Steel (Strength, Ductility, Bond Properties), Prestressing Steel.

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1 Structural Eurocodes and their objectives

For the realization of the European single market, the Commission of the European Communities (CEC) has initiated the work of establishing a set of unified technical rules for the design of building and civil engineering works, which will gradually replace the different rules in force in the various EC-Member States. These technical rules, which became known as the Structural Eurocodes shall lead to structures, which fulfil the following fundamental requirements, established in [1]:

"Basic requirements

A structure shall be designed and executed in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economical way sustain all actions and influences likely to occur during execution and use and remain fit for the use for which it is required.

A structure shall be designed to have adequate:

- Structural resistance
- Serviceability and
- Durability.

In the case of fire the load-bearing capacity of the structure shall be assured for the required period of time."}

In other words, the fundamental requirements, which shall be met, are adequate performance in use, appropriate degree of reliability, adequate performance in service conditions and adequate durability during the design working life (Table 1.). The relationship between these requirements and the economical aspects should be noted.

The Structural Eurocodes provide the technical tools for the achievement of these requirements. The corresponding elements of the design concept are described in the following. They are related to Classes 4 and 5 in Table 1., where the design working life is defined as follows [1]:

"... The design working life is the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary."

2 European standards system for concrete structures

Figure 1 presents the actual European Standards System for building and civil engineering works in concrete, which still consists mainly of European Prestandards (ENV). They are actually converted to European Standards, which will replace the corresponding national standards in force in the various CE-Member States. It should be noted that – according to [2] – this system will be used for "CE-Marking" of construction products (e. g. reinforcing steel) so that they can be used without restriction within the European single market.

In this European Standards System, which provides all elements for structural and durability design four levels can be distinguished:

- Level 1 comprises standards for structural safety [1] and actions on structures; in particular, in [1] basic reliability and durability requirements are established.

- Level 2 consists of Eurocode 2 [3] for the design and detailing of concrete structures.
• Level 3 gives data for structural materials, in particular for concrete [4], for reinforcing steel [5] and the execution of concrete structures [6].

• Level 4 consists of standards for the testing of materials. Most of them are ISO-Standards.

In the following, Eurocode 2 Part 1-1 [3] and its implications for reinforcing steel will be described in more detail. The list of contents in [3] is shown in Figure 2.

Table 1. Indication of the design working life required in [1]

<table>
<thead>
<tr>
<th>Design working life category</th>
<th>Indicative design working life (years)</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>Temporary structures (1)</td>
</tr>
<tr>
<td>2</td>
<td>10-25</td>
<td>Replaceable structural parts, e.g. gantry girders, bearings</td>
</tr>
<tr>
<td>3</td>
<td>15-30</td>
<td>Agricultural and similar structures</td>
</tr>
<tr>
<td>4</td>
<td>50</td>
<td>Building structures and other common structures</td>
</tr>
<tr>
<td>5</td>
<td>100</td>
<td>Monumental building structures, bridges and other civil engineering structures</td>
</tr>
</tbody>
</table>

Note 1: Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.

3 Reliability verification at the ultimate limit states

When – according to Section 2 in Eurocode 2 – considering a limit state of rupture or excessive deformation of a section, member or connection, it shall be verified that:

\[ E_d \leq R_d \] (3.1)

where:

\( E_d \) is the design value of the effect of actions such as internal force, moment or vector representing several internal forces or moments;

\( R_d \) is the design value of the corresponding resistance, associating all structural properties with the respective design values.

For each critical load case, the design values of the effects of actions \( (E_d) \) shall be determined by combining the values of action that are considered to occur simultaneously with expressions in which:

\( A_d \) is the design value of the accidental action;

\( A_{Ed} \) is the design value of seismic action \( A_{Ed} = \gamma_1 A_{Ek} \).
Figure 1. Structure of the European Standards System for Concrete Structures
**Foreword**

1. General
2. Basis of design
3. Materials
4. Durability and cover to reinforcement
5. Structural analysis
6. Ultimate limit states
7. Serviceability limit states
8. Detailing of reinforcement – General
9. Detailing of members and particular rules
10. Additional rules for precast concrete elements and structures
11. Lightweight aggregate concrete structures
12. Plain and lightly reinforced concrete structures

Informative and normative annexes

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Figure 2. List of contents of the new draft for Eurocode 2 [3]

$A_{Ek}$ is the characteristic value of the seismic action;

$G_{k,j}$ is the characteristic value of permanent action $j$;

$P$ is the relevant representative value of a prestressing action;

$Q_{k,1}$ is the characteristic value of the leading variable action 1;

$Q_{k,i}$ is the characteristic value of the accompanying variable action $i$;

$\gamma_{G,j}$ is the partial factor for permanent action $j$;

$\gamma_1$ is an important factor depending on the design situation considered;

$\gamma_p$ is the partial factor for prestressing actions;

$\gamma_{Q,i}$ is the partial factor for variable action $i$;

"+" implies “to be combined with”;

$\Sigma$ implies “the combined effect of”.
In accordance with equation (3.1), the combination of effects of actions to be considered should be based on the design value of the leading variable action and the design combination values of accompanying variable actions:

\[ E_d = E \{ \gamma_{G,j} G_{k,j}; \gamma_p P; \gamma_{Q,1} Q_{k,1}; \gamma_q \Psi_{0,i} Q_{k,i} \} \]  

(3.2)

The combination of actions in brackets in (3.2) may either be expressed as:

\[ \sum_{j=1}^{\gamma} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i=1}^{\gamma} \gamma_q \Psi_{0,i} Q_{k,i} \]  

(3.3)

or alternatively for limit states of rupture, the more unfavourable of the two following expressions:

\[ \sum_{j=1}^{\gamma} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i=1}^{\gamma} \gamma_q \Psi_{0,i} Q_{k,i} \]  

(3.4)

\[ \sum_{j=1}^{\gamma} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i=1}^{\gamma} \gamma_q \Psi_{0,i} Q_{k,i} \]  

(3.5)

where:

\[ \zeta_j \]  

is a reduction factor for unfavourable permanent actions which is less than 1.

The recommended values \( \gamma_F \) for actions are given in Table 2.

The design resistance \( R_d \) in equation (3.1) is expressed in the following form:

\[ R_d = \frac{1}{\gamma_{Rd}} R \left\{ X_{k,i}; \frac{a_d}{\gamma_m} \right\} = \frac{1}{\gamma_{Rd}} R \left\{ \frac{X_{k,i}}{\gamma_m}; \frac{a_d}{\gamma_m} \right\} \]  

(3.6)

where:

\( \gamma_{Rd} \)  

is a partial factor covering uncertainty in the resistance model, plus geometric deviations if these are not modelled explicitly;

\( X_{k,i} \)  

is the characteristic value of a material property \( i \);

\( X_{d,i} \)  

is the design value of material property \( i \);

\( a_d \)  

is a design value of geometrical data (e.g. cross-sectional dimensions, dimensions of members or elements).

Provided that the resistance is a linear function of material strength, the following simplification of expression (3.6) may be made:

\[ R_d = R \left[ \frac{f_{ck}}{\gamma_c}; \frac{f_{yk}}{\gamma_y}; \frac{f_{pk}}{\gamma_p} \right] \]  

(3.7)

where:

\( f_{ck}, f_{yk}, f_{pk} \)  

is the characteristic strength of concrete, reinforcing steel and prestressing steel respectively;
\( \gamma_c, \gamma_s \) are the partial safety factors for concrete and reinforcing/prestressing steel respectively;

\( \alpha \) is a coefficient taking account of long-term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied.

Information about the characteristic strength of concrete and respectively steel will be presented in Section 8.

The partial safety factors, \( \gamma_c \) and \( \gamma_s \) for materials are given in Table 3.

**Table 2.** Recommended values, \( \gamma_F \), for actions in [1]

<table>
<thead>
<tr>
<th>Persistent and transient design situation</th>
<th>Permanent actions ( G_k )</th>
<th>Leading variable or accidental actions ( Q_{k,1} )</th>
<th>Accompanying variable actions ( Q_{k,i} )</th>
<th>Prestress</th>
</tr>
</thead>
<tbody>
<tr>
<td>unfavourable</td>
<td>favourite</td>
<td>action</td>
<td>(if any)</td>
<td>Generally</td>
</tr>
<tr>
<td>( \gamma_F = )</td>
<td>1.35</td>
<td>1.00</td>
<td>1.50</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**Table 3.** Recommended values, \( \gamma_c \) and \( \gamma_s \) for materials in [3]

<table>
<thead>
<tr>
<th>Material</th>
<th>Concrete</th>
<th>Reinforcing and Prestressing Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_{M,i} = )</td>
<td>( \gamma_c = 1.5 )</td>
<td>( \gamma_s = 1.15 )</td>
</tr>
</tbody>
</table>

According to [1] the following ultimate limit states (ULS) shall be verified:

a) loss of equilibrium of the structure or any part of it, considered as a rigid body;
b) failure by excessive deformation, transformation of the structure or any part of it into a mechanism; rupture, loss of stability of the structure or any part of it, including supports and foundations.
c) failure caused by fatigue or other time-dependent effects.

With regard to b) in [3] distinction is made between the following ULS:

- Bending of beams and slabs with or without normal force ([3], Section 6.1)
- Shear (Section 6.2)
- Torsion (Section 6.3)
- Punching (Section 6.4), see Figure 3.
- Design of discontinuity regions with strut-and-tie models (6.5)
- Anchorages and laps (6.6)
- Partially loaded areas (6.7)

The design of slender compression members including second order effects is covered by Section 5.8, the verification for fatigue (see c) above) is subject of Section 6.8 in [3].
In order to satisfy the reliability requirements given by eq. 3.1 at the ULS described above, reinforcing steel used for reinforced concrete shall have the following properties:

- Adequate yield strength, $f_{yk}$, and tensile strength, $f_{tk}$, (Sections 5.8 and 6.2 to 6.8 in [3])
- Surface characteristics which allow the development of the design bond strength, $f_{bd}$, (Section 6.6)
- Adequate fatigue strength $f_{s, fat}$, (Section 6.8)

Numerical values for these properties will be described in Section 8 below.

4 Verification format at the serviceability limit states

At the serviceability limit states (SLS) it shall be verified that:

$$E_d \leq C_d$$  \hspace{1cm} (4.1)

where:

- $C_d$ is the limiting design value of the relevant serviceability criterion (e.g. crack width, deflection or rotation, stress in concrete and/or steel);
- $E_d$ is the design value of the effects of actions specified in the serviceability criterion, determined on the basis of the most unfavourable of the combinations.
The combination of actions to be taken into account in the relevant design situations should be appropriate for the serviceability requirements and performance criteria being verified. In [1] and [3] the following distinction is made:

a) Characteristic combination for limit states:

\[ \sum_{j=1} \gamma_{k,j} \cdot P \cdot \gamma_{i,1} \cdot Q_{k,i} \cdot \psi_{0,i} \cdot \gamma_{k,i} \]  

\( (4.2) \)

b) Frequent combination for limit states:

\[ \sum_{j=1} \gamma_{k,j} \cdot P \cdot \gamma_{1,1} \cdot Q_{k,1} \cdot \sum_{i=1} \psi_{2,i} \cdot Q_{k,i} \]  

\( (4.3) \)

c) Quasi-permanent combination for limit states:

\[ \sum_{j=1} \gamma_{k,j} \cdot P \cdot \sum_{i=1} \psi_{2,i} \cdot Q_{k,i} \]  

\( (4.4) \)

For the representative value of the Prestressing action (i. e. \( P_k \) or \( P_m \)), reference should be made to the relevant design Eurocode, e. g. Eurocode 2 for the type of prestress under consideration. For the values of the combination factor \( \psi \) for buildings, see Table 4.

**Table 4.** Values of \( \psi \) factors for buildings [1]

<table>
<thead>
<tr>
<th>Action</th>
<th>( \psi_0 )</th>
<th>( \psi_1 )</th>
<th>( \psi_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imposed loads in buildings, category (see [1])</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Category A: domestic, residential areas</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category B: office areas</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category C: congregation areas</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category D: shopping areas</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category E: storage areas</td>
<td>1.0</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td>Category F: traffic area, vehicle weight ( \leq 30) KN</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category G: traffic area, vehicle weight ( 30 &lt; \leq 160) KN</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category H: roofs</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Snow loads on buildings</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Finland, Iceland, Norway, Sweden</td>
<td>0.70</td>
<td>0.50</td>
<td>0.20</td>
</tr>
<tr>
<td>- Remainder of CEN Member States, for sites located at altitude H &gt; 1000 m a.s.l.</td>
<td>0.70</td>
<td>0.50</td>
<td>0.20</td>
</tr>
<tr>
<td>- Remainder of CEN Member States, for sites located at altitude H ( \leq 1000) m a.s.l.</td>
<td>0.50</td>
<td>0.20</td>
<td>0</td>
</tr>
<tr>
<td>Wind loads on buildings</td>
<td>0.6</td>
<td>0.2</td>
<td>0</td>
</tr>
<tr>
<td>Temperature (non-fire) in buildings</td>
<td>0.6</td>
<td>0.5</td>
<td>0</td>
</tr>
</tbody>
</table>

The SLS, which must be checked for reasons of adequate performance in service conditions and/or for durability are described in Section 6 below.
5 Structural analysis

5.1 General

According to [3], the purpose of analysis is to establish the distribution of either internal forces and moments, or stresses strains and displacements, over the whole or part of a structure. Additional local analysis shall be carried out where necessary.

Analyses are carried out using idealizations of both the geometry and the behaviour of the structure. The idealizations selected shall be appropriate to the problem being considered.

In the context of Eurocode 2 [3], the common idealizations of the behaviour used for analysis are:

- Elastic behaviour
- Elastic behaviour with limited redistribution
- Plastic behaviour including strut and tie models
- Non-linear behaviour.

Additional local analyses may be necessary where the assumption of linear strain distribution is not considered valid, e. g.

- Supports
- Under concentrated loads
- Beam and beam-column intersections
- Anchorage zones
- Changes in section.

The design concept in Eurocode 2 [3] is based on the requirement that brittle failure of a structure or of parts thereof shall be avoided. Consequently, in the structural analysis, due consideration shall be given to an adequate rotation capacity, \( \Theta_{pl} \), of "plastic hinges" (see Figure 4).

![Figure 4](image)

**Figure 4.** Formation of a "plastic hinge" in a reinforced concrete section

The rotation capacity, \( \Theta_{pl} \), depends in fact on several parameters. However, the most important ones are:
• The "ductility" properties of the reinforcing steel in terms of the ratio \( \frac{f_t}{f_y} \) and the elongation of maximum load, \( \varepsilon_u \) (see Section 8.3 and Figure 7).

• The ultimate load bearing capacity of the concrete in the "plastic" hinge.

Both criteria are part of the verification methods below.

5.2 Linear analysis

The relevant important design rules in [3] may be summarized as follows:

Linear analysis of elements based on the theory of elasticity may be used for both the serviceability and ultimate limit states. For the determination of the action effects of loads, linear analysis generally assumes uncracked cross sections, linear stress-strain relationships and mean values of the elastic modulus.

For effects of imposed deformations at the ultimate limit state a reduced stiffness corresponding to the full cracked sections may be assumed. For the serviceability limit state a gradual evolution of cracking should be considered.

However, linear analysis applied for ultimate limit states requires careful detailing of the reinforcement to cover all zones where tensile stresses may appear.

5.3 Linear analysis with limited distribution

Linear analysis with limited redistribution may be applied to the analysis of beams and frames for the verification of the Ultimate Limit States (ULS).

In continuous beams where the ratio of adjacent spans is \( 0.5 < \frac{l_1}{l_2} < 2 \), in beams of non-sway frames and in elements subject predominantly to flexure (including slabs) and where \( \delta \) is the ratio of the final moment to the original moment, the conditions given below should be satisfied:

(I) With reinforcement of Class B and Class C (see Section 8.3 and Table 15.)

\[
\delta \geq 0.64 + 0.8 \left( \frac{x}{d} \right) \geq 0.70
\]

for concrete grades not greater than C50/60

\[
\delta \geq 0.72 + 0.8 \left( \frac{x}{d} \right) \geq 0.80
\]

for concrete grades C55/67 and C60/75

(II) With reinforcement of Class A

\[
\delta \geq 0.64 + 0.8 \left( \frac{x}{d} \right) \geq 0.85
\]

for concrete grades not greater than C50/60

\[
\delta = 1
\]

for concrete grades C55/67 and C60/75
5.4 Plastic methods of analysis

According to Section 5.6 in [3], methods based on plastic analysis shall only be used for the design at ULS. In any case, the plastic rotation capacity must be checked. Indirect actions (imposed or restrained deformations) need only to be considered if a significant part of the plastic range in the moment-curvature-diagram is used for the redistribution of the indirect action effects.

The plastic analysis is either based on the lower bound (static) method or on the upper bound (kinematic) method. The static method includes: the strip method for slabs, the strut and tie approach for deep beams, corbels, anchorages, walls and plates loaded in their plane. The kinematic method includes: yield hinges method for beams, frames and one way slabs; yield lines theory for slabs.

The effects of previous applications of loading may generally be ignored and a monotonic increase of the intensity of actions may be assumed.

When analysing beams and frames, the allowable rotations $\Theta_{pl}$ for reinforcing steel classes A, B or C and for concrete grades up to C50/60 ($\varepsilon_{c2u} = 0.0035$) are given by expressions (5.5) to (5.10). For concrete grades C55/67 and C60/75, these values for $\Theta_{pl}$ have to be reduced with the factor $\left| \varepsilon_{c2u} \right| / 0.0035$ where $\left| \varepsilon_{c2u} \right|$ is the ultimate concrete strain.

- Reinforcement of Class C
  
  for $0.05 \leq x/d \leq 0.14$ \quad $\Theta_{pl} = 4.740 \cdot \left| \varepsilon_{c2u} \right| \cdot e^{3.738 \cdot (x/d)}$ (5.5)
  
  for $0.14 < x/d \leq 0.50$ \quad $\Theta_{pl} = 13.020 \cdot \left| \varepsilon_{c2u} \right| \cdot e^{-3.480 \cdot (x/d)}$ (5.6)

- Reinforcement of Class B
  
  for $0.05 \leq x/d \leq 0.16$ \quad $\Theta_{pl} = 2.718 \cdot \left| \varepsilon_{c2u} \right| \cdot e^{4.644 \cdot (x/d)}$ (5.7)
  
  for $0.16 < x/d \leq 0.50$ \quad $\Theta_{pl} = 9.768 \cdot \left| \varepsilon_{c2u} \right| \cdot e^{-3.351 \cdot (x/d)}$ (5.8)

- Reinforcement of Class A
  
  for $0.05 \leq x/d \leq 0.16$ \quad $\Theta_{pl} = 0.834 \cdot \left| \varepsilon_{c2u} \right| \cdot e^{6.301 \cdot (x/d)}$ (5.9)
  
  for $0.16 < x/d \leq 0.50$ \quad $\Theta_{pl} = 2.851 \cdot \left| \varepsilon_{c2u} \right| \cdot e^{-1.382 \cdot (x/d)}$ (5.10)

In these expressions denote:

- $x/d$ is the relative depth of neutral axis at ULS
- $\varepsilon_{c2u}$ depends on $f_{ck}$ and varies between $\varepsilon_{c2u} = - 3.5 \%$ for $f_{ck} \leq 50 \text{ N/mm}^2$ and $\varepsilon_{c2u} = -2.6 \%$ for $f_{ck} = 90 \text{ N/mm}^2$ (i. e. C90/105).

A graphic – approximately linear – presentation of the above formulas is given in Figure 5.

In slabs, adequate rotation capacity may be assumed if reinforcing steel of class B or C is used and if the area of tensile reinforcement does not exceed, at any point or in any direction, a value corresponding to $x/d = 0.25$. 
5.5 Non-linear analysis

In the context of EC 2, non-linear methods of analysis may be used for both ULS and SLS, provided that equilibrium and compatibility are satisfied and an adequate non-linear behaviour for materials is assumed. The analysis can be first or second order.

In [3] detailed information on the practical application of non-linear analysis is not provided. It is recommended to make reference to appropriate literature.

6 Durability requirements

According to [1] a structure shall be designed in such a way that deterioration over its design working life shall not impair the durability and performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance. In this respect, the protection of the reinforcement against corrosion due to carbonation or chlorides is an important aspect.

In order to achieve an adequately durable structure, the following should be taken into account:
- The intended or foreseeable use of the structure;
- The required performance criteria;
- The expected environmental conditions;
- The composition, properties and performance of the materials; e.g. reinforcing steel;
- The choice of the structural system;
- The shape of members and the structural detailing;
- The quality of workmanship, and the level of control;
- The particular protective measures;
- The maintenance during the design working life.

Information on these items is given in the individual parts of the European Standards System (see Figure 1).

The above requirements to be met by concrete structures depend mainly on the environment to which the concrete structure is exposed. Environment in this context implies chemical and physical actions resulting in effects, which are not considered as loads in structural design. The environmental actions defined in [4] are shown in Tables 5 and 6, where a rough distinction is made between six deterioration mechanisms for concrete and steel respectively.

The actions in Tables 5 and 6 may, where relevant, be considered as local or micro conditions. Local conditions are those around the structure after having been built, taking into account the specific actions where the structure or the structural element is located (e.g. relative humidity RH, CO₂-content).

However, in some circumstances, micro conditions need to be considered. These denote environmental actions on a specific surface of a structural element. This may, for example, apply to the following circumstances:

- Exposition to driving rain
- Exposition to sun radiation
- Contact with earth, ground water, seawater etc.
<table>
<thead>
<tr>
<th>Deterioration mechanism</th>
<th>Class designation</th>
<th>Description of the environment</th>
<th>Informative examples where exposure classes may occur</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 No risk of corrosion or attack</td>
<td>X0</td>
<td>Very dry</td>
<td>Concrete inside buildings with very low humidity (RH &lt; 45%)</td>
</tr>
<tr>
<td>2 Steel corrosion induced by carbonation</td>
<td>XC1</td>
<td>Dry</td>
<td>Concrete inside buildings with low humidity (RH &lt; 65%)</td>
</tr>
<tr>
<td></td>
<td>XC2</td>
<td>Wet, rarely dry</td>
<td>Parts of water retaining structures, many foundations</td>
</tr>
<tr>
<td></td>
<td>XC3</td>
<td>Moderate humidity (RH &lt; 80%)</td>
<td>Concrete inside buildings with moderate or high air RH; external concrete sheltered from rain</td>
</tr>
<tr>
<td>3 Steel corrosion induced by chlorides</td>
<td>XC4</td>
<td>Cyclic wet and dry</td>
<td>Surfaces subject to water contact, not within class XC2</td>
</tr>
<tr>
<td>4 Steel corrosion induced by chlorides from sea water</td>
<td>XD1</td>
<td>Moderate humidity</td>
<td>Concrete surfaces exposed to direct spray containing chlorides</td>
</tr>
<tr>
<td></td>
<td>XD2</td>
<td>Wet, rarely dry</td>
<td>Swimming pools; concrete exposed to industrial water containing chlorides</td>
</tr>
<tr>
<td></td>
<td>XD3</td>
<td>Cyclic wet and dry</td>
<td>Parts of bridges; pavements; car park slabs</td>
</tr>
<tr>
<td></td>
<td>XS1</td>
<td>Exposed to air-borne salt, not in direct contact with sea water</td>
<td>Structures near to or on the coast</td>
</tr>
<tr>
<td></td>
<td>XS2</td>
<td>Submerged</td>
<td>Parts of marine structures</td>
</tr>
<tr>
<td></td>
<td>XS3</td>
<td>Tidal, splash and spray zones</td>
<td>Parts of marine structures</td>
</tr>
<tr>
<td>5 Freeze/thaw attack on concrete</td>
<td>XF1</td>
<td>Moderate water saturation, no de-icing agents</td>
<td>Vertical concrete surfaces exposed to rain and freezing</td>
</tr>
<tr>
<td></td>
<td>XF2</td>
<td>Moderate water saturation, with de-icing agents</td>
<td>Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents</td>
</tr>
<tr>
<td></td>
<td>XF3</td>
<td>High water saturation, no de-icing agents</td>
<td>Horizontal concrete surfaces exposed to rain and freezing</td>
</tr>
<tr>
<td></td>
<td>XF4</td>
<td>High water saturation, with de-icing agents</td>
<td>Road and bridge decks exposed to de-icing agents and vertical concrete surfaces exposed to direct spray containing de-icing agents and freezing</td>
</tr>
<tr>
<td>6 Chemical attack on concrete</td>
<td>XA1, XA2, XA3</td>
<td>Aggressive chemical environment</td>
<td>See Table 6.</td>
</tr>
</tbody>
</table>

With regard to the resistance of concrete structures against environmental actions, the choice of a durable concrete requires consideration of its composition and may result in a high compressive strength. Indicative strength classes depending on the environmental exposure classes defined in Tables 5 and 6 are given in Table 7.
Table 6. Limiting values for exposure classes XA for chemical attack in [4]

<table>
<thead>
<tr>
<th>Chemical characteristic</th>
<th>XA1</th>
<th>XA2</th>
<th>XA3</th>
<th>Test method</th>
</tr>
</thead>
<tbody>
<tr>
<td>SO$_2^+$ mg/l in water</td>
<td>$\geq 200$ and $\leq 600$</td>
<td>$&gt; 600$ and $\leq 3000$</td>
<td>$&gt; 3000$ and $\leq 6000$</td>
<td>EN 196-2</td>
</tr>
<tr>
<td>total amount SO$_2^+$ mg/kg in soil$^{(1)}$</td>
<td>$\geq 2000$ and $\leq 3000^{(2)}$</td>
<td>$&gt; 3000^{(2)}$ and $\leq 12000$</td>
<td>$&gt; 12000$ and $\leq 24000$</td>
<td>EN 196-2$^{(2)}$</td>
</tr>
<tr>
<td>ph of water</td>
<td>$\leq 6.5$ and $\geq 5.5$</td>
<td>$&lt; 5.5$ and $\geq 4.5$</td>
<td>$&lt; 4.5$ and $\geq 4.0$</td>
<td>DIN 4030-2</td>
</tr>
<tr>
<td>Acidity of soil</td>
<td>$&gt; 20^\circ$ Baumann Gully</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CO$_2$ mg/l aggressive in water</td>
<td>$\geq 15$ and $\leq 40$</td>
<td>$&gt; 40$ and $\leq 100$</td>
<td>$&gt; 100$</td>
<td></td>
</tr>
<tr>
<td>NH$_4^+$ mg/l in water</td>
<td>$\geq 15$ and $\leq 30$</td>
<td>$&gt; 30$ and $\leq 60$</td>
<td>$&gt; 60$ and $\leq 100$</td>
<td>ISO 7150-1</td>
</tr>
<tr>
<td>Mg$^{2+}$ mg/l in water</td>
<td>$\geq 300$ and $\leq 1000$</td>
<td>$&gt; 1000$ and $\leq 3000$</td>
<td>$&lt; 3000$</td>
<td>ISO 7980</td>
</tr>
</tbody>
</table>

Footnotes:
1. Clay soils with a permeability below $10^{-5}$ m/s may be moved into a lower class.
2. The 3000 mg/kg limit is reduced to 2000 mg/kg, where there is a risk of accumulation of sulphate ions in the concrete due to drying and wetting cycling or capillary suction.
3. The test method prescribes the extraction of SO$_2^+$ by hydrochloric acid; alternatively, water extraction may be used, if experience is available in the place of use of the concrete.

Table 7. Indicative Strength Classes

<table>
<thead>
<tr>
<th>Exposure Classes according to Table 5, and 6, respectively</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrosion of reinforcement</td>
</tr>
<tr>
<td>Carbonation-induced corrosion</td>
</tr>
<tr>
<td>XC1</td>
</tr>
<tr>
<td>XC1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Indicative Strength Class</th>
<th>C20/25</th>
<th>C25/30</th>
<th>C 30/37</th>
<th>C30/37</th>
<th>C35/45</th>
<th>C30/37</th>
<th>C35/45</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete attack</td>
<td>No risk</td>
<td>Freeze / Thaw attack</td>
<td>Chemical attack</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>X0</td>
<td>XF1</td>
<td>XF2</td>
<td>XF3</td>
<td>XA1</td>
<td>XA2</td>
<td>XA3</td>
<td></td>
</tr>
<tr>
<td>C12/15</td>
<td>C30/37</td>
<td>C25/30</td>
<td>C30/37</td>
<td>C30/37</td>
<td>C35/45</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

According to Eurocode 2, [3], a nominal concrete cover to reinforcement shall be introduced in the design calculations. It is given by:

$$\text{nom } c = \min c + \Delta c$$  \hspace{1cm} (6.1)

where:

- $\text{nom } c$ denotes the nominal cover;
- $\min c$ is the minimum cover;
- $\Delta c$ is an allowance for tolerances.
For the determination of the *minimum* concrete cover, \( \text{min } c \), the following criteria apply:

- Safe transmission of bond forces
- Avoidance of spalling
- Adequate fire resistance
- The protection of the steel against corrosion.

In the latter case, the protection against corrosion depends upon the continuing presence of a surrounding alkaline environment provided by an adequate thickness of good quality, well-cured concrete. In the absence of other provisions, adequate thickness may be assumed if the values of \( \text{min } c \) given in Table 8. for normal weight concrete are used. In any case the *nominal* value of cover to reinforcement should be such that excessive corrosion of the steel is avoided.

**Table 8.** Minimum cover requirements for normal weight concrete

<table>
<thead>
<tr>
<th>Environmental Requirement</th>
<th>Exposure Classes according to Table 5. and 6.</th>
<th>Corrosion of reinforcement</th>
<th>Chloride-induced corrosion</th>
<th>Chloride-induced corrosion from seawater</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No risk</td>
<td>Carbonation-induced corrosion</td>
<td>Chloride-induced corrosion</td>
<td>Chloride-induced corrosion from seawater</td>
</tr>
<tr>
<td></td>
<td>X0</td>
<td>XC1</td>
<td>XC2</td>
<td>XC3</td>
</tr>
<tr>
<td>( \text{c}_{\text{min}} ) Reinforcing steel(^1)</td>
<td>10</td>
<td>15</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>( \text{c}_{\text{min}} ) Precast-sing steel(^1)</td>
<td>20</td>
<td>25</td>
<td>35</td>
<td>40</td>
</tr>
</tbody>
</table>

**Bond Requirement**

\[
\text{c}_{\text{min}} \geq \Phi \text{ or } \Phi_{n} \\
\text{c}_{\text{min}} \geq (\Phi + 5\text{mm}) \text{ or } (\Phi_{n} + 5\text{mm}) \text{ if } d_{g} > 32\text{mm} \\
\text{(where: } \Phi \text{ is the diameter of the bar, the wire, the strand or the duct; } \Phi_{n} \text{ is the equivalent diameter for a bundle and } d_{g} \text{ is the nominal maximum aggregate size)}
\]

Notes:

1) The minimum concrete cover for slabs and for structural elements which have a strength class two strength classes higher than indicated in Table A1 of [3] (except for exposure class XC1) may be reduced by 5mm providing there an adequate number of sufficiently stiff spacers. Other relationships between minimum cover and concrete quality may be given in a National Annex.

2) In extreme cases, special protective measures against corrosion may be required (e.g. stainless steel reinforcement).

The design tolerance in expression (6.1) should normally be \( \Delta c = 10\text{mm} \). However, in certain cases, \( \Delta c \) may be reduced. This applies to situations where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover and non conforming members are rejected (especially in the case of precast elements). In these cases, the allowance in design for tolerances \( \Delta c \) may be reduced:

\[
\Delta c_{\text{red}} = \Delta c - x \quad (\Delta c > x > 0) \quad (6.2)
\]

In any case, the *nominal* value of cover to reinforcement should be such that excessive corrosion of the steel is avoided.
7 Verification of the Serviceability limit states

7.1 General

The serviceability limit states shall be those that concern:

- The functioning of the structure or structural elements under normal use
- The comfort of people
- The appearance of the construction works.

However, in [1], [3] the term “appearance” is concerned with such criteria as high deflection and extensive cracking, rather than aesthetics.

In [3], the verification of serviceability limit states are based on criteria concerning the following aspect:

a) Stress limitation ([3], Section 7.2)
b) Crack control (Section 7.3)
c) Deflection control (Section 7.4).

The respective design provisions are summarized in the following, where regard is given to the relevant properties of reinforcing steel.

7.2 Limitation of stresses

Excessive compressive stress in the concrete under the service load may promote the formation of longitudinal cracks and lead to micro-cracking in the concrete or higher than linearly predicted levels of creep. If the proper functioning of a member is likely to be adversely affected by these (e.g. corrosion), measures shall be taken to limit the stresses to an appropriate level.

Longitudinal cracks may occur if the stress level under the characteristic combination of loads exceeds a critical value. Such cracking may lead to a reduction in durability. In the absence of other measures, such as an increase in cover of reinforcement in the compressive zone or confinement by transverse reinforcement, it may be appropriate to consider limiting the compressive stress to 0.6 $f_{ck}$ in areas exposed to environments of exposure classes XD, XF and XS (see Table 5.).

If the stress in concrete under the quasi-permanent loads is lower than 0.45 $f_{ck}$, linear creep can be assumed. If the stress in concrete exceeds 0.45 $f_{ck}$, non linear creep should be considered ([3], Section 3.1.3).

Stresses in the reinforcing bars under serviceability conditions which could lead to inelastic deformation of the steel, shall be avoided as this will lead to large, permanently open, cracks. This requirements will be met provided that, under the characteristic combination of loads the tensile stress in ordinary reinforcement does not exceed 0.8 $f_{yk}$. Where the stress is due only to imposed deformations, a stress of 1.0 $f_{yk}$ will be acceptable. The stress in prestressing tendons should not exceed 0.75 $f_{pk}$ after allowance for losses, where $f_{pk}$ denotes the maximum tensile strength.
7.3 Crack control

The durability of concrete structures may adversely be affected by excessive cracking. Besides that, cracking shall be limited to a level that will not impair the proper functioning of the structure or cause its appearance to be unacceptable.

For common types of cracks in concrete structures two primary causes may be distinguished:

- Cracks caused by the rheological properties of the fresh or hardening concrete
- Cracks caused by loading and/or imposed deformations

The first type of cracks can be controlled by appropriate measures of concrete technology, in particular by the composition of the concrete mix, proper placing and curing. Corresponding rules are provided in [4], [6].

For the control of cracks caused by loading and/or imposed deformation, the design concept in Eurocode 2 provides two basic tools:

- The requirement of a minimum bonded steel reinforcement
- The limitation of crack width

It should be noted, however, that effective crack width control depends to a large extent on the bond behaviour between concrete and reinforcing steel. Therefore, in the context of Eurocode 2 [3] it is anticipated that concrete and reinforcing bars meet the requirements in Table 14. and Table 15. of this paper. Otherwise, the provisions below need to be adjusted.

The minimum steel reinforcement has two functions: it should ensure an equilibrium at the time when cracks may first be expected. In addition, the area of the minimum reinforcement should be such that crack widths with an unacceptable value are avoided. In most cases, the minimum reinforcement is calculated for imposed deformations due to the dissipation of the hydration heat, i.e. for a concrete age between 3 to 5 days after casting. It depends mainly on the actual concrete tensile strength, \( f_{ct} \).

Unless a more rigorous calculation shows lesser areas to be adequate, the required minimum areas of reinforcement may be calculated from:

\[
A_s \sigma_s + \xi_1 A_p \Delta \sigma_p = k_c k f_{ct,eff} A_{ct} \quad (7.1)
\]

Where:

- \( A_s \) area of reinforcing steel within tensile zone
- \( A_p \) area of prestressing steel within an area of not more than 300 mm around the steel reinforcement in the tensile zone
- \( \xi_1 \) adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel:
  \[
  \xi_1 = \sqrt[\phi_s / \phi_p]{\xi}
  \]
- \( \phi_s \) largest diameter of reinforcing steel
- \( \phi_p \) equivalent diameter of prestressing steel
  \[
  \phi_p = 1.60 \sqrt{A_p} \quad \text{for tendons with several strands or wires}
  \]
\[ \phi_p = 1.75 \phi_{wire} \] for single strands with 7 wires

\[ \phi_p = 1.20 \phi_{wire} \] for single strands with 3 wires

\[ \zeta \] ratio of bond strength of prestressing steel and high bond reinforcing steel. In the absence of appropriate data, \( \zeta \) may be taken from Table 9.

\[ A_{ct} \] 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.

\[ \sigma_s \] the maximum stress permitted in the reinforcing steel 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 (Table 10.) or the maximum bar spacing (Table 11.).

\[ \Delta \sigma_p \] stress increase in prestressing steel from zero stress in the concrete at the same level the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur \( (f_{ct,eff} = f_{ctm}) \). In many cases, such as where the dominant imposed deformation arises from dissipation of the heat of hydration, this may be within 3-5 days from casting depending on the environmental conditions, the shape of the member and the nature of the formwork. Values of \( f_{ct,eff} = f_{ctm} \) may be obtained from [3] by taking as the class the strength at the time cracking is expected to occur. When the time of cracking cannot be established with confidence as being less than 28 days, it is suggested that a minimum tensile strength of 3 MPa is adopted or its value based on the relevant indicative strength class according to Table 8.

\[ k_c \] a coefficient which takes account of the nature 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 rectangular sections and webs of box sections and T-sections:

\[
k_c = 0.4 \left[ 1 + \frac{\sigma_c}{k_1 (h/h^*) f_{ct,eff}} \right] \leq 1 \quad \text{(7.2)}
\]

for flanges of box sections and T-section:

\[
k_c = 0.9 \frac{F_{cr}}{A_{ct} f_{ct,eff}} \geq 0.5 \quad \text{(7.3)}
\]

\[ \sigma_c \] mean stress of the concrete acting on the part of the section under consideration \( (\sigma_c < 0 \text{ for compression force}) \):

\[ \sigma_c = \frac{N_{Ed}}{bh} \]

\( N_{Ed} \) axial force at the serviceability limit state acting on the part of the cross-section under consideration (compressive force negative). \( N_{Ed} \) should be determined considering the characteristic values of prestress and axial forces under the quasi-permanent combination of actions

\[ h^* \]

\[ h^* = h \quad \text{for } h < 1.0 \text{ m} \]

\[ h^* = 1.0 \text{ m} \quad \text{for } h \geq 1.0 \text{ m} \]

\[ k_1 \] a coefficient considering the effects of axial forces on the stress distribution:

\[ k_1 = 1.5 \quad \text{if } N_{Ed} \text{ is a compressive force} \]

\[ k_1 = \frac{2h^*}{3h} \quad \text{if } N_{Ed} \text{ is a tensile force} \]

\[ F_{cr} \] tensile force within the flange immediately prior to cracking due to the cracking moment calculated with \( f_{ct,eff} \)

\[ k \] coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces:

\[ k = 1.0 \quad \text{for webs with } h \leq 300 \text{ mm or flanges with widths less than 300 mm} \]

\[ k = 0.65 \quad \text{for webs with } h \geq 800 \text{ mm or flanges with widths greater than 800 mm} \]

Intermediate values may be interpolated.
Table 9. Nominal ratio $\xi$ of mean bond stress of prestressing steel and high bond reinforcing steel for crack control

<table>
<thead>
<tr>
<th>Type of Tendon</th>
<th>Pre-tensioned members</th>
<th>Post-tensioned members</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth prestressing steel</td>
<td>-</td>
<td>0.4</td>
</tr>
<tr>
<td>Strands</td>
<td>0.6</td>
<td>0.5</td>
</tr>
<tr>
<td>Ribbed prestressing wires</td>
<td>0.8</td>
<td>0.7</td>
</tr>
<tr>
<td>Ribbed prestressing bars</td>
<td>1.0</td>
<td>0.8</td>
</tr>
</tbody>
</table>

For the limitation of crack width, the value design $w_k$ may be obtained from the relation:

$$w_k = s_{\text{max}} (\varepsilon_{sm} - \varepsilon_{cm})$$  \hspace{1cm} (7.4)

where:

- $w_k$: design crack width
- $s_{\text{max}}$: maximum crack spacing
- $\varepsilon_{sm}$: mean strain in the reinforcement, under the relevant combination of loads, taking into account the effects of tension stiffening, etc.
- $\varepsilon_{cm}$: mean strain in concrete between cracks

$\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from the expression:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - 0.4 f_{\text{eff}} (1 + \alpha \rho_{p,\text{eff}})}{\rho_{p,\text{eff}} \varepsilon_s} \geq 0.6 \frac{\sigma_s}{E_s}$$  \hspace{1cm} (7.5)

where

- $\alpha$: ratio $E_s / E_{ci}$

The maximum final crack spacing can be calculated, in mm, from the expression:

$$s_{\text{r,max}} = \frac{\sigma_s}{3.6 \rho_{p,\text{eff}}} \leq \frac{\sigma_s}{3.6 f_{\text{cl,eff}}}$$  \hspace{1cm} (7.6)

For simplification and where at least the minimum reinforcement given by expression (7.1) is provided, crack widths will not generally be excessive if:

- for cracking caused dominantly by restraint, the bar sizes given in Table 10. are not exceeded where the steel stress is the value obtained immediately after cracking [i.e. $\sigma_s$ in Expression (7.1)]
- for cracks caused dominantly by loading, either the provisions of Table 10. or the provisions of Table 11. are complied with

For prestressed concrete sections, the stresses in the reinforcement should be calculated regarding the prestress as an external force without allowing for the stress increase in the tendons due to loading.
Table 10. Maximum bar diameters $\phi_s^*$ for high bond bars

<table>
<thead>
<tr>
<th>Steel stress [N/mm²]</th>
<th>$w_k = 0.4$ mm</th>
<th>$w_k = 0.3$ mm</th>
<th>$w_k = 0.2$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>40</td>
<td>32</td>
<td>25</td>
</tr>
<tr>
<td>200</td>
<td>32</td>
<td>25</td>
<td>26</td>
</tr>
<tr>
<td>240</td>
<td>20</td>
<td>16</td>
<td>12</td>
</tr>
<tr>
<td>280</td>
<td>16</td>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>320</td>
<td>12</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>360</td>
<td>10</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>400</td>
<td>8</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>450</td>
<td>6</td>
<td>5</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 11. Maximum bar spacing for high bond bars

<table>
<thead>
<tr>
<th>Steel stress [N/mm²]</th>
<th>$w_k = 0.4$ mm</th>
<th>$w_k = 0.3$ mm</th>
<th>$w_k = 0.2$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>300</td>
<td>300</td>
<td>200</td>
</tr>
<tr>
<td>200</td>
<td>300</td>
<td>250</td>
<td>150</td>
</tr>
<tr>
<td>240</td>
<td>250</td>
<td>200</td>
<td>100</td>
</tr>
<tr>
<td>280</td>
<td>200</td>
<td>150</td>
<td>50</td>
</tr>
<tr>
<td>320</td>
<td>150</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>360</td>
<td>100</td>
<td>50</td>
<td>-</td>
</tr>
</tbody>
</table>

For reinforced concrete the maximum bar diameter may be modified as follows:

\[
\phi_s = \phi_s^* \left(\frac{f_{ct,eff}}{2.5}\right) \frac{h_{cr}}{10(h-d)} \geq \phi_s^* \left(\frac{f_{ct,eff}}{2.5}\right) \quad \text{for restraint cracking} \tag{7.7}
\]

\[
\phi_s = \phi_s^* \frac{h_{cr}}{10(h-d)} \geq \phi_s^* \quad \text{for load induced cracking} \tag{7.8}
\]

where:
- $\phi_s$ is the adjusted maximum bar diameter
- $\phi_s^*$ is the maximum bar size given in Table 10.
- $h$ is the overall depth of the section
- $h_{cr}$ is the depth of the tensile zone immediately prior to cracking, considering the characteristic values of prestress and axial forces under the quasi-permanent combination of actions
- $d$ is the effective depth to the centroid of the outer layer of reinforcement

7.4 Limitation of deformation
The deformation of a member or structure should not be such that it adversely affects its proper functioning or appearance. Appropriate limiting values of deflection taking into account the nature of the structure, of the finishes, partitions and fixings and upon the function of the structure should be agreed with the client.

The appearance and general utility of the structure may be impaired when the calculated sag of a beam, slab or cantilever subjected to the quasi-permanent loads exceeds span/250. The sag is assessed relative to the supports. Precamber may be used to compensate for some or all of the deflection but any upward deflection incorporated in the formwork should not generally exceed span/250.

However, in buildings, it is generally not necessary to calculate the deflections explicitly as simple rules, such as limits to span/depth ratio may be formulated which will be adequate for avoiding deflection problems in normal circumstances. More rigorous checks are necessary for members which lie outside such limits or where deflection limits other than those implicit in simplified methods are appropriate.

Provided that reinforces concrete beams or slabs in buildings are dimensioned so that they comply with the limits of span to depth given in this clause, their deflections should not normally exceed the limits set out before. The limiting span/depth ratio is obtained by taking a basic ratio from Table 12, and multiplying this by correction factors to allow for the type of reinforcement used and other variables. No allowance has been made for any precamber in the derivation of these tables.

Table 12. Basic ratios of span/effective depth for reinforced concrete members without axial compression

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Concrete highly stressed</th>
<th>Concrete lightly stressed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported beam, one or two-way spanning simply supported slab</td>
<td>14</td>
<td>20</td>
</tr>
<tr>
<td>End span of continuous beam or one way continuous slab or two-way spanning slab continuous over long side</td>
<td>18</td>
<td>26</td>
</tr>
<tr>
<td>Interior span of beam or one-way or two-way spanning slab</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Slab supported on columns without beams (flat slab) (based on longer span)</td>
<td>17</td>
<td>24</td>
</tr>
<tr>
<td>Cantilever</td>
<td>6</td>
<td>8</td>
</tr>
</tbody>
</table>

8 Material Data

8.1 General

From the previous Sections it can be concluded that the design concept of Eurocode 2 [3] requires adequate material properties in terms of strength, bond, workability and deformation characteristics. The relevant provisions are summarized below.
8.2 Concrete

Eurocode 2 [3] covers normal-weight and heavy-weight concrete as well as light-weight aggregate concrete. The strength classes for normal-weight and heavy-weight concrete are shown in Table 13. Generally, the compressive strength of concrete is classified by concrete strength classes which relate to the characteristic (5 %) cylinder strength $f_{ck}$ or the cube strength $f_{ck,\text{cube}}$, in accordance with [4]. However, the design of resistance $R_d$ is based on the cylinder strength, denoted as $f_{ck,\text{cylinder}}$ or for reasons of simplicity, $f_{ck}$. Table 13. shows also the mean value, $f_{c\text{tm}}$, and the 5 % fractile, $f_{ck,0.05}$, of the axial tensile strength of concrete.

The design value of the ultimate bond stress, $f_{bd}$, for ribbed bars may be taken as:

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot \frac{f_{ck,0.05}}{\gamma_c}$$

(8.1)

where $f_{ck,0.05}$ is the 5 % fractile of concrete tensile strength according Table 13. and $\gamma_c$ the partial safety coefficient for concrete. Due to the increasing brittleness of higher strength concrete, $f_{ck,0.05}$ should be limited here to the value of C60, unless it can be verified that the average bond strength increases above this limit.

$\eta_1$ is a coefficient related to the quality of the bond condition and the position of the bar during concreting (see Figure 6)

$\eta_1 = 1.0$ when "good conditions" are obtained and $\eta_1 = 0.7$ for all other cases and for bars in structural elements built with slip-forms, unless it can be shown that "good" bond conditions exist

$\eta_2$ is related to the bar diameter:

$\eta_2 = 1.0$ for $\phi \leq 32$ mm

$\eta_2 = (132 - \phi) / 100$ for $\phi > 32$ mm

---

**Figure 6.** Description of bond conditions
Values for $f_{bd}$ are given in Table 14.

Other design values for concrete in [3] (e.g. modulus of elasticity, creep and shrinkage coefficients) are approximately identical with those in CEB/FIP-Model 1990 [7].

**Table 13.** Strength classes for normal-weight and heavy-weight concrete in EN 206 [4]

<table>
<thead>
<tr>
<th>Strength class</th>
<th>$f_{ck, \text{cylinder}}$ [N/mm²]</th>
<th>$f_{ck, \text{cube}}$ [N/mm²]</th>
<th>$f_{ctm}$ [N/mm²]</th>
<th>$f_{ctk,0.05}$ [N/mm²]</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>C 12/15</td>
<td>12</td>
<td>15</td>
<td>1.6</td>
<td>1.1</td>
<td>Normal strength concrete</td>
</tr>
<tr>
<td>C 16/20</td>
<td>16</td>
<td>20</td>
<td>1.9</td>
<td>1.3</td>
<td></td>
</tr>
<tr>
<td>C 20/25</td>
<td>20</td>
<td>25</td>
<td>2.2</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>C 25/30</td>
<td>25</td>
<td>30</td>
<td>2.6</td>
<td>1.8</td>
<td></td>
</tr>
<tr>
<td>C 30/37</td>
<td>30</td>
<td>37</td>
<td>2.9</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>C 35/45</td>
<td>35</td>
<td>45</td>
<td>3.2</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>C 40/50</td>
<td>40</td>
<td>50</td>
<td>3.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>C 45/55</td>
<td>45</td>
<td>55</td>
<td>3.8</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>C 50/60</td>
<td>50</td>
<td>60</td>
<td>4.1</td>
<td>2.9</td>
<td></td>
</tr>
<tr>
<td>C 55/67</td>
<td>55</td>
<td>67</td>
<td>4.2</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>C 60/75</td>
<td>60</td>
<td>75</td>
<td>4.4</td>
<td>3.1</td>
<td></td>
</tr>
<tr>
<td>C 70/85</td>
<td>70</td>
<td>85</td>
<td>4.6</td>
<td>3.2</td>
<td></td>
</tr>
<tr>
<td>C 80/95</td>
<td>80</td>
<td>95</td>
<td>4.8</td>
<td>3.4</td>
<td></td>
</tr>
<tr>
<td>C 90/105</td>
<td>90</td>
<td>105</td>
<td>5.0</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>C 100/115</td>
<td>100</td>
<td>115</td>
<td>5.2</td>
<td>3.6</td>
<td></td>
</tr>
</tbody>
</table>

**Table 14.** Design values of the ultimate bond stress, for good bond conditions, $f_{bd}$, and for other cases, $f'_{bd}$, as function of $f_{ck}$ for reinforcing bars with $\varnothing \leq 32$ mm.

<table>
<thead>
<tr>
<th>$f_{ck}$ [N/mm²]</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
<th>55</th>
<th>$\geq$ 60</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{bd}$ [N/mm²]</td>
<td>1.65</td>
<td>1.95</td>
<td>2.25</td>
<td>2.70</td>
<td>3.00</td>
<td>3.30</td>
<td>3.75</td>
<td>4.05</td>
<td>4.35</td>
<td>4.50</td>
<td>4.65</td>
</tr>
<tr>
<td>$f'<em>{bd} = 0.7 f</em>{bd}$ [N/mm²]</td>
<td>1.15</td>
<td>1.36</td>
<td>1.57</td>
<td>1.89</td>
<td>2.10</td>
<td>2.31</td>
<td>2.62</td>
<td>2.83</td>
<td>3.04</td>
<td>3.15</td>
<td>3.25</td>
</tr>
</tbody>
</table>

### 8.3 Reinforcing Steel

In Eurocode 2, the behaviour of reinforcing steel is specified by the following properties (see Figure 7):

- Yield strength ($f_{yk}$ or $f_{0.2k}$)
- Tensile strength ($f_t$)
- Ductility ($\varepsilon_u$ and $f_t/f_{yk}$)
- Bendability
- Bond characteristics ($f_R$)
- Section sizes and tolerances
- Fatigue
- Weldability

The values required in [3] are summarized in Table 15.

![Figure 7. Typical stress-strain diagrams of reinforcing steel](image)

In Table 15., with regard to structural analysis (see Section 5), three classes of ductility are defined: Class A, B and C. Where non-linear or plastic methods of analysis are applied, only high ductility steel (classes B or C) shall be used. Where other reinforcement is used it shall be demonstrated that it complies with the requirements given in [3]. Table 16. shows a comparison of the basic properties of reinforcing steel in Eurocode 2 [3] and the Chinese Standard GB/T 1499-98: "Hot rolled ribbed steel bars for the reinforcement of concrete".
Table 15. Properties of recommended in [3]

<table>
<thead>
<tr>
<th>Product form</th>
<th>Bars and de-coiled rods</th>
<th>Wire Fabrics</th>
<th>Requirement or quantile value (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class</td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Characteristic yield strength (f_y) (MPa)</td>
<td>500</td>
<td>450 or 500</td>
<td>500</td>
</tr>
<tr>
<td>((f_y/f_t)k)</td>
<td>≥1,05</td>
<td>≥1,08</td>
<td>≥1,15 &lt;1,35</td>
</tr>
<tr>
<td>Total elongation at maximum force, (\varepsilon_u) (%)</td>
<td>2,5</td>
<td>5,0</td>
<td>7,5</td>
</tr>
<tr>
<td>(f_{y,act}) (MPa)</td>
<td>650</td>
<td>540 or 650</td>
<td>650</td>
</tr>
<tr>
<td>Fatigue stress range(^2) ((N = 2 \times 10^7)) (MPa)</td>
<td>150</td>
<td>100</td>
<td>10,0</td>
</tr>
<tr>
<td>Bendability</td>
<td>Rebond test(^1)</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Shear strength (%)</td>
<td>-</td>
<td>0,3 A (f_y)</td>
<td></td>
</tr>
<tr>
<td>Bond(^3)</td>
<td>Nominal bar size (mm)</td>
<td>0,035</td>
<td>min. 5,0</td>
</tr>
<tr>
<td>Projected rib factor, (f_{R,min})</td>
<td>5-6</td>
<td>0,04</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6,5 to 12</td>
<td>0,056</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt;12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deviation from nominal mass ((individual bar or wire)) (%)</td>
<td>± 4,5</td>
<td>max. 5,0</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. The rebend test must be carried out in accordance with EN 10080 using a mandrel size no greater than that specified for bending in Table 8.1 of this standard. In order to check bendability a visual check shall be carried out after the first bend.
2. If higher values are shown by testing and approved by an appropriate authority, the design values in Table 6.3 in [3] may be modified. Such testing should be in accordance with EN 10080.
3. Where it can be shown that sufficient bond strength is achievable with \(f_R\) values less than specified above the values may be relaxed. In order to ensure sufficient bond strength is achieved, the bond stresses must satisfy expressions (a) and (b) when tested using the CEB/RILEM beam test:

\[
\tau_m \geq 0,098 (80-1,2 \varnothing)  
\]

\[
\tau_r \geq 0,098 (130-1,9 \varnothing) 
\]

where:

\(\varnothing\) = the nominal bar size (mm)
\(\tau_m\) = mean value of bond stress (MPa) at 0,01; 0,1 and 1mm slip
\(\tau_r\) = the bond stress at failure by slipping
Table 16. Comparison of the requirements for ribbed reinforcing bars in Eurocode 2 [3] and in the Chinese Standard GB/T 1499-98

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Product form</td>
<td>Bars and de-coiled rods</td>
<td>Ribbed bars</td>
</tr>
<tr>
<td>Class</td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Characteristic yield strength $f_{yk}$ or $f_{y2k}$ (MPa)</td>
<td>500</td>
<td>450 or 500</td>
</tr>
<tr>
<td>$(f/f_{yk})_k$</td>
<td>$\geq 1.05$</td>
<td>$\geq 1.08$</td>
</tr>
<tr>
<td>Total elongation at maximum force, $\varepsilon_u$ (%)</td>
<td>2.5</td>
<td>5.0</td>
</tr>
<tr>
<td>$f_{y,\text{act}}$ (MPa)</td>
<td>650</td>
<td>540 or 650</td>
</tr>
</tbody>
</table>

For the design of concrete structures, the following assumptions apply:

Design should be based on the nominal cross-section area of the reinforcement and the design value derived from the characteristic values.

$f_{yk}$ for normal design, either of the following assumptions may be made (see Figure 8):

a) an inclined top branch with a strain limit of $\varepsilon_{ud}$ and a maximum stress of $k f_{yk} / \gamma_s$ at $\varepsilon_{uk}$, where $k = (f_t/f_{yk})_k$

b) a horizontal top branch without the need to check the strain limit. The recommended value is $0.9 \varepsilon_{uk}$.

Figure 8. Idealised and design stress-strain diagrams for reinforcing steel (for tension and compression)
The mean value of density may be assumed to be 7850 kg/m$^3$ and the design value of the modulus of elasticity, $E_s$ may be assumed to be 200 GPa.

References


