Eurocodes 2: Concrete, reinforced concrete and prestressed concrete structures

EC 2: Concrete Structures. Overview – Basic Design Concept

EC 2: Ouvrages en béton. Exposé – Principes de base pour le calcul

EC 2: Betontragwerke. Übersicht – Allgemeines Bemessungskonzept

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SUMMARY

Eurocode 2 is the main part of the European Regulation System for the design and execution of buildings and civil engineering works in plain, reinforced and prestressed concrete. The basic elements of the design concept in EC 2 are presented.

RESUME

L'Eurocode 2 constitue l'élément principal du futur système de réglementation pour le calcul et l'exécution des bâtiments et des ouvrages de génie civil en béton, béton armé et béton précontraint. Les principes de base de l'EC 2 pour la vérification des structures sont présentés.

ZUSAMMENFASSUNG

Eurocode 2 ist der zentrale Bestandteil des künftigen europäischen Regelwerks für den Entwurf und die Ausführung von Tragwerken aus unbewehrtem Beton, Stahl- oder Spannbeton. Die Hauptelemente des Bemessungskonzepts werden erläutert.

1 EUROCODE 2 AND THE EUROPEAN REGULATION SYSTEM FOR CONCRETE STRUCTURES

Eurocode 2 [1] is part of the future European Regulation System for the design of buildings and civil engineering works in plain, reinforced and prestressed concrete (Fig. 1). It is concerned with the essential requirements for resistance, serviceability and durability of concrete structures. Execution is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules.

The work on EC2 started in 1979 and was originally based on the CEB/FIP Model Code 1978 [2]. A first important step was the publication a first draft for EC2 [3] in 1984, issued in form of a Technical Report. The CEC-Member States were invited to comment on it. In 1985, about 1500 pages of partly very detailed comments have been received. They were assessed in 1986 and 1987 by the Editorial Group for Eurocode 2 chaired by Professor Franco LEVI (Italy). At the end of 1989, a revised final Draft for EC2 was approved by this Group and submitted to Sub-Committee 2 (SC2) of TC250 of CEN which was formed in 1990. After a slight editorial improvement of this Draft, EC2 was issued in form of a European Pre-Standard ENV at the end of 1991 [1].

EC2 Part 1 therefore is the result of a sound discussion of more than 10 years on a European level involving numerous specialists in the specific areas. It can be therefore assumed that EC2 Part 1 reflects to a large extent the state-of-the-art in the individual CEC-Member States.

2 SCOPE OF EC2 PART 1; DEVELOPMENT OF FURTHER PARTS

EC2 Part 1 (Fig. 2) gives the *general* basis for the design of buildings and civil engineering works in reinforced and prestressed concrete made with normal weight aggregates. In addition, Part 1 gives detailed rules which are mainly applicable to ordinary buildings. The applicability of these rules may be limited, for practical reasons or due to simplifications. The use of the relevant rules and any limits of applicability are explained in the text where necessary.

In particular, Part 1 of EC2 does actually not cover

- the resistance to fire;
- particular aspects of special types of buildings (such as tall buildings);
- particular aspects of special types of civil engineering works (e.g. viaducts, bridges, dams, pressure vessels, off-shore platforms or liquid-retaining structures);
- no-fines and aerated concrete elements, and those made with heavy aggregate or containing structural steel (future Eurocode 4).

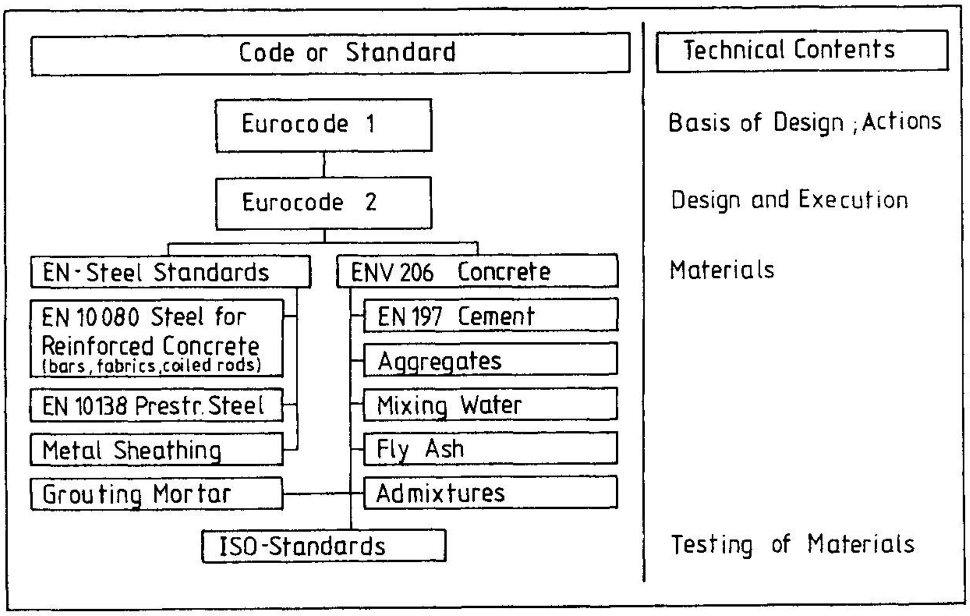

Fig. 1: Future Regulation System for Concrete Structures

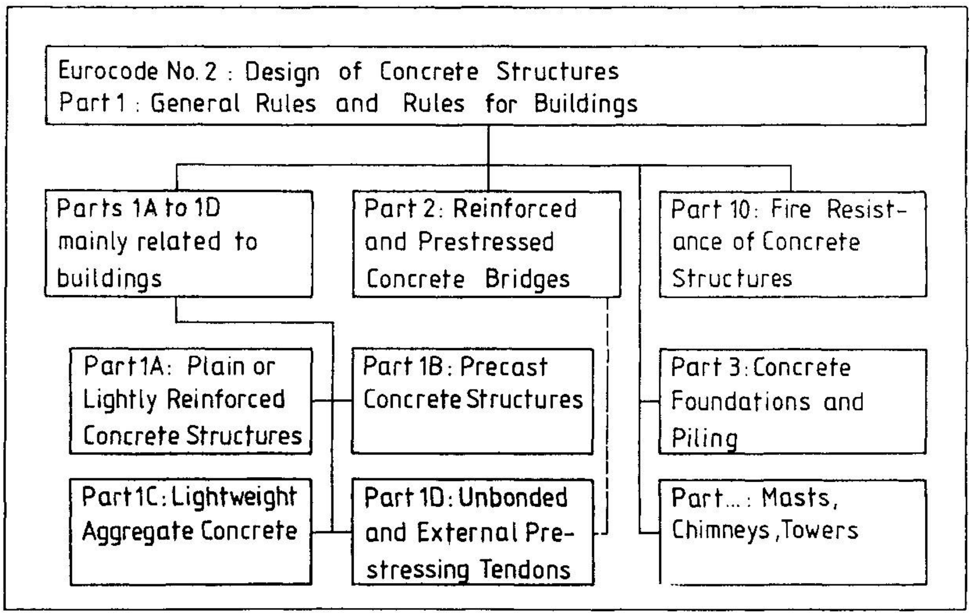

Fig. 2: Future Structure of Eurocode 2

Due to this limited scope of Part 1, EC2 will be supplemented by further Parts which will complement or adapt it for particular aspects of special types of building or civil engineering works, special methods of construction and for certain other aspects of design which are of general practical importance.

Further Parts of EC2 are actually being prepared by CEN/TC250/SC2 on the basis of mandates in the following areas (Fig. 2):

Part 1A: Plain or lightly reinforced concrete structures;
Part 1B: Precast concrete elements and structures;
Part 1C: The use of lightweight aggregate concrete;
Part 1D: The use of unbonded and external prestressing tendons;
Part 10: Fire resistance of concrete structures;
Part 2: Reinforced and prestressed concrete bridges.

These Parts will be issued in form of European Pre-Standards (ENV) in 1993 (Part 1A - 1D, Part 10) and 1994 respectively (Part 2).

High priority is given by CEN/TC205/SC2 to the following Parts of EC2 which will hopefully be included in the working programme for 1993 and 1994:

Part 3: Concrete foundations and piling;
Part 4: Containments and retaining structures;
Part X: Design assisted by testing; material related aspect

as well as a Mandate for the maintenance and further development of EC2 [1].

This demonstrates that the issue of ENV 1992-1-1 [1] is only a first step towards a harmonized European regulation system for concrete structures (Fig. 1) and futher, important steps have to follow.

3 HARMONIZATION PROBLEMS; PRINCIPLES AND RULES FOR APPLICATION; INDICATIVE VALUES

Concrete construction has in all European countries a long tradition. The result is that - even when based on the same physical model - the individual design rules and practices are likely to differ significantly (Fig. 3). The main objective of EC2 therefore was *not* the *total* unification of the design rules but a *gradual* harmonization. This aim was achieved by

- the publication of EC2 in form of a European Pre-Standard (ENV) (Fig. 4);
- the distinction between *Principles* and *Rules for Application*;
- the use of *indicative* numerical values.

The *Principles* comprise:

- general statements and definitions for which there is *no alternative*, as well as

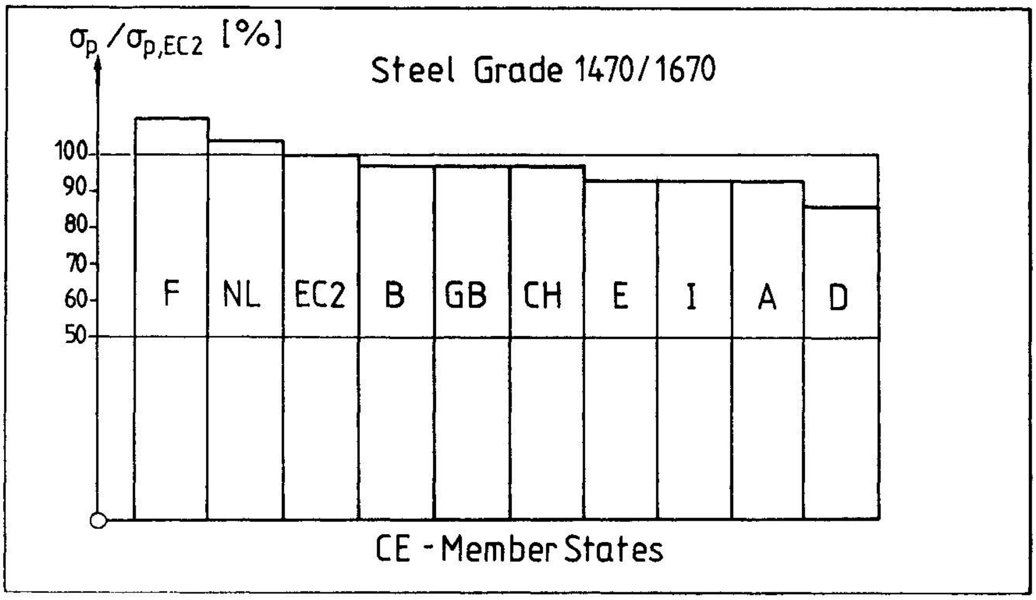

Fig. 3: Comparison of the admissible stresses σ_p in prestressing tendons

7.6 Implementation

7.6.1 Members shall make the ENV available at national level in an appropriate form promptly and announce its existence in the same way as for EN/HD.

7.6.2 Existing conflicting national standards may be kept in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

Fig. 4: CEN-Rules for the implementation of European Pre-Standards (ENV)

- requirements and analytical models for which no alternative is permitted unless specificalyl stated.

The *Application Rules* are generally recognised rules which follow the Principles and satisfy their requirements. However, it is permissible to use *alternative* rules different from the Application Rules in EC2, provided that it can be shown that the alternative rules accord with the relevant Principles and that they are at least equivalent with regard to the resistance, serviceability and durability achieved with the present Eurocode 2.

A second tool for the gradual harmonization of design rules is the use of indicative values, e.g. of numerical values identified by ⌴ in the text (Fig. 5). Other values may be specified by the CEN Member States, for example in the National Application Documents (NAD, see Section 5).

4 GENERAL DESIGN CONCEPT - LIMIT STATES; DURABILITY REQUIREMENTS

According to the "Model Chapter 2.1" common to all Eurocodes, structures shall be designed and constructed in such a way that they are suited to their intended use throughout their anticipated service life, taking economic aspects into account.

Consequently, concrete structures shall

- sustain all mechanical actions with an adequate degree of reliability and

- be adequately resistant to chemical, biological, climatic and similar actions.

In their intended use, concrete structures shall also

- with an adequate degree of reliability sustain specified actions in serviceability conditions - without a decrease in their utility.

These general requirements concerning the ultimate bearing capacity and serviceability also include durability. They are quantified in EC2-Chapter

4 Section and Member Design

in particular in Sub-Chapters (see Fig. 6)

4.1 Durability Requirements
4.3 Ultimate Limit States

and

4.4 Serviceability Limit States.

Additional informations for the avoidance of damages by hazards to an extent disproportionate to the original cause are subject of Chapter

1.3 Assumptions

P(3) Numerical values identified by ⊔ are given as indications. Other values may be specified by Member States.

Example: Admissible Prestressing Force

$$F_p = \sigma_{pm0} \cdot A_p = \boxed{0,75} \cdot f_{pk} \cdot A_p \text{ or } \boxed{0,85} \cdot f_{p0.1k} \cdot A_p$$

Fig. 5: The use of indicative values

1 Introduction

2 Basis of Design
- 2.1 Fundamental Requirements
- 2.2 Definition and Classification
- 2.3 Design Requirements
- 2.4 Durability
- 2.5 Analysis

3 Material Properties
- 3.1 Concrete
- 3.2 Reinforcing Steel
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- 3.4 Prestressing Devices

4 Section and Member Design
- 4.1 Durability Requirements
- 4.2 Design Data
- 4.3 Ultimate Limit States
- 4.4 Serviceability Limit States

5 Detailing Provisions

6 Construction and Workmanship

7 Quality Control

Appendices

Fig. 6: Contents list of EC2 Part 1

5 Detailing Provisions

and in particular of Sub-Chapter

5.5 Limitation of Damage Due to Accidental Actions

which contains rules for the design and detailing of tie systems.

The Ultimate Limit States (ULS) covered by EC2-Chapter 4.3 include the

4.3.1 ULS for Bending and Longitudinal Force
4.3.2 Shear
4.3.3 Torsion (including combined effects of actions)
4.3.4 Punching
4.3.5 ULS Induced by Structural Deformations (Buckling).

In these ULS, it shall be verified that

$$S_d\ [\Sigma\ \gamma_G * G_k + \gamma_Q * Q_{k,1} + \sum_{i>1} \gamma_Q * \Psi_0 * Q_{k,i} + \gamma_P * P_k]$$

$$\leq R_d\ \left[\frac{f_{ck}}{\gamma_c}; \frac{f_{yk}}{\gamma_s}; \frac{f_{pk}}{\gamma_s} \right] \qquad (1)$$

where

S_d	design value of an internal force or moment
R_d	corresponding design resistance
G_k	characteristic value of permanent actions
$Q_{k,1}$	characteristic value of one of the variable actions
$Q_{k,i}$	characteristic value of the other variable actions
P_k	characteristic value of prestressing force
$\gamma_G, \gamma_Q, \gamma_P$	partial safety coefficient for permanent actions, variable actions and for the actions due to prestress
Ψ_0	combination factor
f_{ck}, f_{yk}, f_{pk}	characteristic strength of concrete, reinforcing steel and prestressing steel respectively
γ_c, γ_s	partial safety coefficient for concrete and steel.

Values for the coefficients γ_G, γ_Q, γ_P, γ_c and γ_s are shown in Table 1. with regard to imposed deformations Q_{IND}, where non-linear methods of analysis are used, the factors for variable actions Q_k given in Table 1 apply. For a linear calculation, these factors for unfavorable effects should be reduced by 20 % (i.e. $\gamma_{IND} = \gamma_Q = 1,2$).

Values for the combination factor Ψ_0 will be found in the future Eurocode 1 "Basis of Design and Actions on Structures".

In the Serviceability Limit States (SLS), it shall be verified that

$$E_d \le C_d \tag{2}$$

or

$$E_d \le R_d \tag{3}$$

where

C_d denotes a nominal value or a function of design properties of the concrete structure under consideration

E_d in the design effect of actions, determined on the basis of the relevant load combination, e.g. of the rare, frequent or quasi-permanent combination of load.

Table 1: Safety coefficients for fundamental combinations

Safety coefficient for	unfavorable effect	favorable effect
1	2	3
permanent actions G_k	$\gamma_G = 1,35$	$\gamma_G = 1,0$
variable actions Q_k	$\gamma_Q = 1,50$	$\gamma_Q = 0$
prestressing force P_k [1)]	$\gamma_P = 1,2$ or $1,0$	$\gamma_P = 0,9$ or $1,0$
concrete	$\gamma_c = 1,50$	---
reinforcing and prestressing steel	$\gamma_s = 1,15$	---

1) these values will be applied according to the relevant clauses in EC2.

A verification according to equations (2) and (3) is necessary in the following Serviceability Limit States:

4.4.1 Limitation of Stresses under Serviceability Conditions
4.4.2 Limit States of Cracking
4.4.3 Limit States of Deformation.

According to Clause 4.4.3.1 in EC2, the appearance, general utility and durability of concrete structures may be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds $l_{eff}/250$ (l_{eff}: effective span). In addition, deflections may cause damage to partitions, to members attached to, or in contact with the member considered if they exceed the value $l_{eff}/500$. Experience shows that these limits may govern design and detailing of structural concrete members mainly subjected to bending.

D u r a b i l i t y is also an important design criterion in EC2. For this reason, Sub-Chapter

4.1 Durability Requirements

was included which summarizes in form of a "Checklist" all parameters which are likely to impair the longterm behaviour of concrete structures. These parameters concern the

- actions, in particular the actions due to the environmental conditions;
- design (cover to reinforcement)
- materials, in particular the composition of concrete;
- construction (curing periods, compaction of the concrete).

In addition to the design rules, Chapters

6 Construction and Workmanship

and

7 Quality Control

provide a series of minimum specification requirements for the standard of workmanship and quality control which must be achieved on site in order to ensure that the design assumptions of EC2 are valid and hence that the intended levels of safety, serviceability and durability will be attained.

5 ASSESSMENT OF THE DESIGN CONCEPT IN EC2; NATIONAL APPLICATIONS

In comparison with the existing design codes in the CEN-Member States, EC2 may lead to more economic solutions (Fig. 7), in particular in design situations where the Ultimate Limit States are predominant.

For a general assessment of the design concept of EC2 it should be noted, however, that design and detailing of concrete structures may be governed either by the ULS or by the SLS (Fig. 8). For this reason a final answer to the question whether or not EC2 leads to more economic results in comparison with the relevant national Codes cannot be given.

For the maintenance and future development of EC2, *p r a c t i c a l* experience is necessary. Forthis reason, the CEN-Member States are requested to apply EC2 on a trial basis and in parallel to their national Codes and Standards.

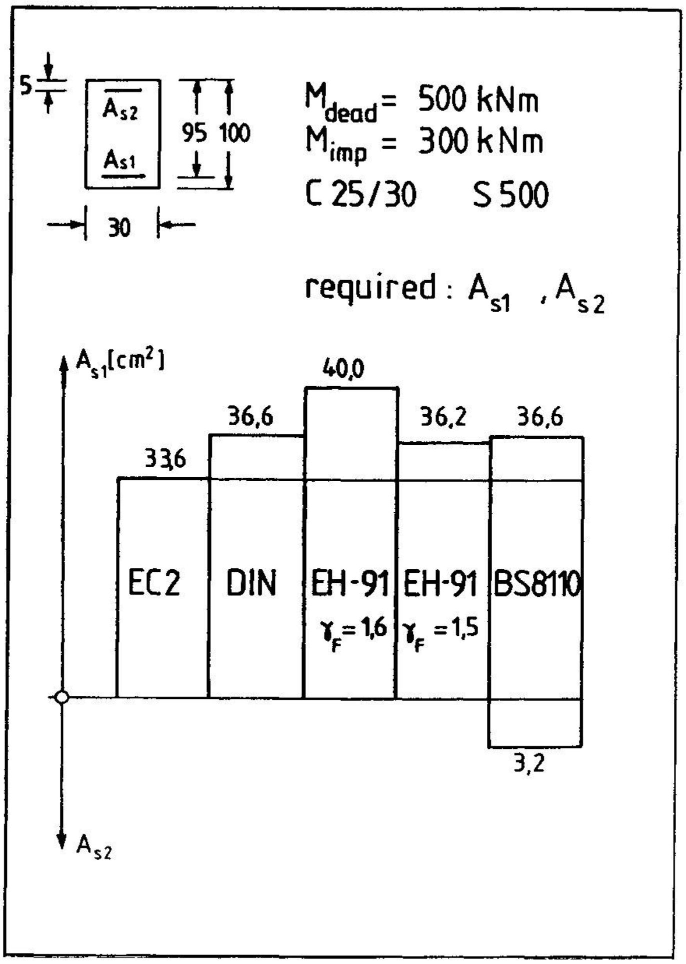

Fig. 7: **Comparison of Design Results according to EC2, DIN 1045, BS 8110 and the Spanish Code EH-91**

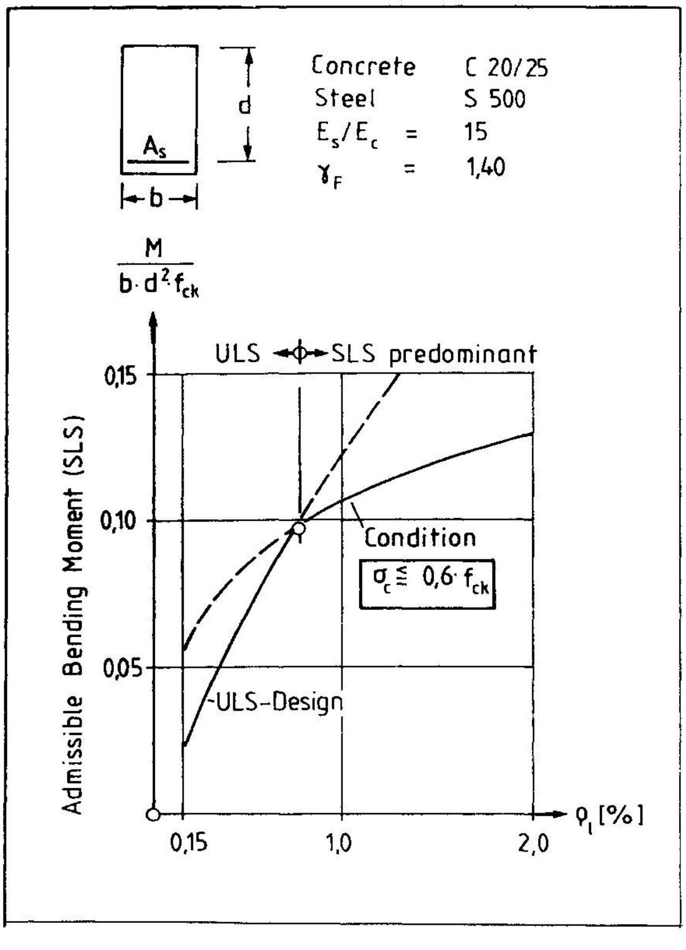

Fig. 8: **Link between ULS- and SLS conditions in the design concept of EC2**

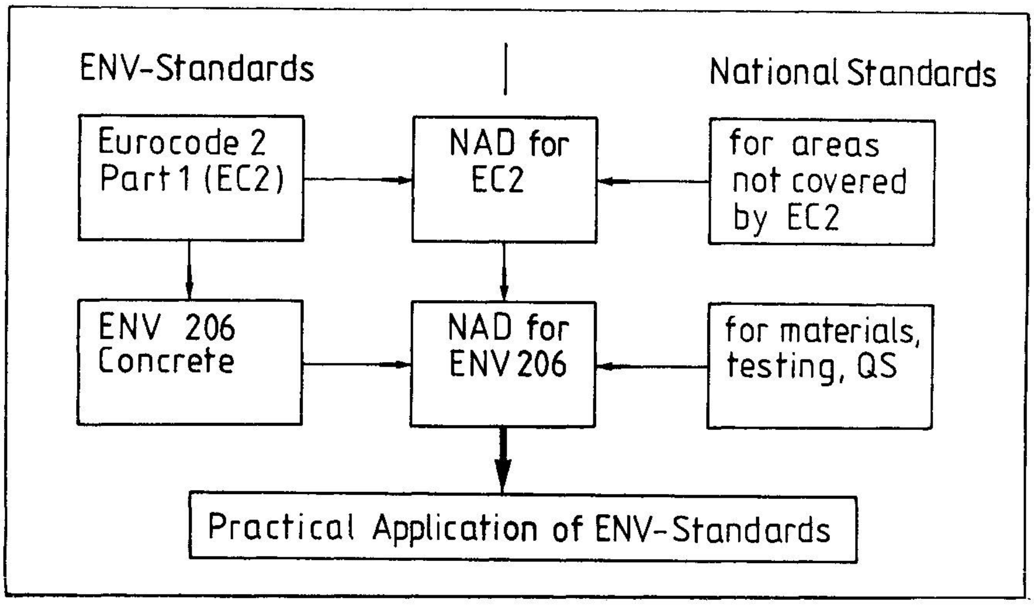

Fig. 9: National Application Documents for the Practical Use of EC2

However, many of the harmonized supporting standards for EC2 (see Fig. 1), such as, for example, Eurocode 1 giving values for actions to be taken into account, will not be available by the time when ENV 1992-1-1 [1] is issued. It is therefore anticipated that National Application Documents (NAD) giving definitive values for safety elements, referencing compatible supporting standards and providing national guidance on the application of this Pre-Standard, will be issued by each member country or its Standards Organisation. The Principle is shown in Fig. 9.

CONCLUSIONS

The issue of Eurocode 2 [1] is a first important step to harmonized European regulations for design and construction of concrete structures. However, for the implementation of the European internal market, further supporting Codes and Standards are necessary. For their development, the input from the profession is needed. Therefore, all engineers are invited to contribute to this within their field of activity.

REFERENCES

[1] ENV 1992-1-1: Eurocode 2: Design of Concrete Structures - Part 1: General Rules and Rules for Buildings (December 1991).

[2] CEB-FIP Model Code for Concrete Structures 1978. CEB-Bulletin d'Information 124/125. Paris 1978.

[3] Commission of European Communities: Eurocode No. 2: Common Unified Rules for Concrete Structures. Report EUR 8848 DE, EN, FR. Brüssel 1984.

EC 2: Concrete Structures – Material Data

EC 2: Structures en béton – Propriétés des matériaux

EC 2: Betontragwerke – Baustoffkennwerte

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SUMMARY

Material data needed for design purposes are the subject of chapters 3 and 4.2 of EC 2 distinguishing between data for concrete, reinforcing steel, prestressing steel and prestressing devices. For material technology, general requirements, testing and quality control, reference is made in EC 2 to European Standards, in particular to ENV 206, prEN 10 080 and prEN 10 138. The main clauses of EC 2 and the above Standards are described in the present article.

RESUME

Les propriétés des matériaux requises pour le dimensionnement font l'objet des chapitres 3 et 4.2 de l'EC 2 qui concernent le béton, l'armature ainsi que les aciers et accessoires de précontrainte. Pour la technologie des matériaux, les exigences générales, les essais et le contrôle de qualité, l'EC 2 se réfère aux normes européennes, en particulier à l'ENV 206 et aux projets des normes prEN 10 080 et prEN 10 138. Les dipsositions les plus importants de l'EC 2 et des normes précitées font l'objet du présent article.

ZUSAMMENFASSUNG

Baustoffkennwerte für die Bemessung sind Gegenstand der Abschnitte 3 und 4.2 von EC 2, wobei zwischen solchen für Beton, Betonstahl, Spannstahl und Spannverfahren unterschieden wird. Hinsichtlich der Baustofftechnologie, entsprechenden allgemeinen Anforderungen, Prüfverfahren und Güteprüfungen nimmt EC 2 Bezug auf Europäische Normen, insbesondere auf ENV 206 sowie auf die Entwürfe von prEN 10 080 und prEN 10 138. Die wichtigsten Festlegungen in EC 2 und den vorgenannten Normen sind im vorliegenden Beitrag zusammengefasst.

1 GENERAL-REFERENCE DOCUMENTS

Material properties are the subject of Chapter 3 of Part 1 of Eurocode 2, subdivided into Sub-Chapters

- 3.1 Concrete
- 3.2 Reinforcing steel
- 3.3 Prestressing steel
- 3.4 Prestressing devices.

Chapter 3 is relatively concise, summarizing only the data for design of concrete structures and referring for production methods, general requirements, testing and quality control to other Standards such as:

- ENV 206 Concrete-Performance, production, placing and compliance criteria (issued 1990);

- prEN 10 080 Steel for the reinforcement of concrete. Weldable ribbed reinforcing steel B 500. Technical delivery conditions for bars, coils and welded fabric (draft 1991);

- prEN 10 138 Prestressing steel
  - Part 1: General requirements (draft 1992)
  - Part 2: Stress relieved cold drawn wire (draft 1992)
  - Part 3: Strand (draft 1992)
  - Part 4: Hot rolled and processed bars (draft 1992)
  - Part 5: Quenched and tempered wire (draft 1992)

Design data of materials are given in Chapter 4.2 of EC2. Lightweight aggregate concrete is considered in ENV 206 but not in Part 1 of Eurocode 2. It is subject of an additional Part 1C which is actually under preparation.

2 CONCRETE
2.1 Technology (ENV 206)

The main topics of ENV 206 are the basic requirements for concrete composition and constituents, the requirements for durability, the properties of fresh and hardened conrete, the specifications for mixes and the requirements for the operations of production (mixing), transport, placing and curing of fresh concrete and finally the quality control procedures to ensure that the specified requirements are satisfied.

The following paragraphs give a survey of those topics which are of direct importance for the designer using Eurocode 2.

Concrete is classified in classes according to its compressive *strength*, based on characteristic strength determined on cylinders or cubes (see section 2.2 "Strength" below).

Durability requirements are related to environmental conditions classified in Table 1: Exposure classes related to environmental conditions (identical with Table 2 of ENV 206 and 4.1 of Eurocode 2).

Table 1: Exposure classes related to environmental conditions

Exposure class		Examples of environmental conditions
1 dry environment		interior of dwellings or offices[1]
2 humid environment	a without frost	- interior of buildings where humidity is high (e.g. laundries) - exterior components - components in non-aggressive soil and/or water
	b with frost	- exterior components exposed to frost - components in non-aggressive soil and /or water and exposed to frost - interior components where the humidity is high and exposed to frost
3 humid environment with frost and de-icing agents		- interior and exterior components exposed to frost and de-icing agents
4 seawater environment	a without frost	- components completely or partially submerged in seawater, or in the splash zone - components in saturated salt air (coastal air)
	b with frost	- components partially submerged in seawater or in the splash zone and exposed to frost - componenets in sturated salt air and exposed to frost
The following classes may occur alone or in combination with the above classes:		
5 aggressive chemical environment[2]	a	- slightly aggressive chemical environment (gas, liquid or solid) - aggressive industrial atmosphere
	b	moderately aggressive chemical environment (gas, liquid or solid)
	c	highly aggressive chemical environment (gas, liquid or solid)

[1] This exposure class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a prolonged period of time

[2] Chemically aggressive environments are classified in ISO 9690. The following equivalent exposure conditions may be used:
Exposure class 5a: ISO classification A1G, A1L, A1S
Exposure class 5b: ISO classification A2G, A2L, A2S
Exposure class 5c: ISO classification A3G, A3L, A3S

Durability requirements for concrete are given in Table 3 of ENV 206, related to the above exposure classes. They concern mainly the maximum water/cement ratio, the minimum cement content and, in case of exposure to frost, also the minimum air content of fresh concrete.

As an example, for the normal conditions in residential buildings, i.e. exposure class 2a, the maximum w/c ratio admissible for reinforced and prestressed concrete is 0.60, the minimum cement content has to be 280 kg/m^3 for reinforced and 300 kg/m^3 for prestressed concrete.

Minimum requirements for *cover* to reinforcement in view of durability are given in Eurocode 2. The minimum values for exposure class 2a are 20 mm for reinforcing steel and 30 mm for prestressing steel.

The important effect of *curing* and of the *quality* of the concrete cover has been recognised during the last years, and got full attention in ENV 206 (Para 10.6). Conditions and methods of curing are given there. Depending on the strength development of the concrete (rapid, medium or slow), on the temperature of concrete during curing and the ambient conditions, Table 12 in ENV 206 gives mimimum curing periods. They vary between 1 day under best conditions up to 10 days under the worst.

Quality control is dealt with in Chapter 11 in ENV 206. It consists of two distinct, but interconnected parts, namely *production control* and *compliance control*.

Production control to be carried out by the contractor, comprises material control, equipment control, control of production procedure and concrete properties. The inspections and tests to be carried out are laid down in corresponding tables (Tables 14 to 17).

Conformity control is done by using one of the following systems:

Case 1 - Verification by a certification body
Case 2 - Verification by the client.

Sampling plans and conformity criteria are laid down in Chapter 11 of ENV 206 for the different possible cases. Conformity criteria for compressive strength may be one of the following:

<u>Criterion 1</u> (for 6 or more consecutive samples):

$$\left.\begin{aligned} \bar{x}_n &\geq f_{ck} + \lambda * s_n \\ x_{min} &\geq f_{ck} - k \end{aligned}\right\} \qquad (1)$$

Values for λ and k may be taken from Table 19 in ENV 206 according to the number of samples n.

<u>Criterion 2</u> (for 3 samples):

$$\left.\begin{aligned} \bar{x}_3 &\geq f_{ck} + 5 \\ x_{min} &\geq f_{ck} - 1 \end{aligned}\right\} \qquad (2)$$

2.2 Strength

Eurocode 2 is based on the characteristic 28 days compressive strength f_{ck} measured on *c y l i n d e r s* and defined as that value of strength below which 5 % of all possible strength test results for the specified concrete may be expected to fall.

Design is based on a strength class of concrete in accordance with ENV 206. These classes are related to the cylinder strength f_{ck} and the cube strength $f_{ck,cube}$, the latter only mentioned as an alternative method to prove compliance.

The tensile strength may be derived from the compressive strength by the equation

$$f_{ctm} = 0{,}30 * f_{ck}^{2/3}, \tag{3}$$

f_{ctm} being the mean tensile strength in uniaxial tension.

For design purposes, also 5% and 95% fractiles of the characteristic tensile strength:

$$\begin{aligned} f_{ctk,0,05} &= 0{,}7 * f_{ctm} \\ f_{ctk,0,95} &= 1{,}3 * f_{ctm} \end{aligned} \tag{4}$$

have to be considered, depending on the problem under consideration.

The values of strength are given in Table 2 (identical with Table 3.1 of EC2). Design values are derived from the characteristic values by applying the appropriate partial safety factor γ_c for concrete, such as:

a) design value of concrete cylinder compressive strength:

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \tag{5}$$

b) the basic design shear strength of members without shear reinforcement (Table 4.8):

$$\tau_{Rd} = \frac{0{,}25*f_{ctk,0,05}}{\gamma_c} \tag{6}$$

c) the design values for the ultimate bond stress for good bond conditions:

$$f_{bd} = \frac{0{,}36\sqrt{f_{ck}}}{\gamma_c} \quad \text{for plain bars} \tag{7}$$

$$f_{bd} = \frac{2{,}25*f_{ctk,0,05}}{\gamma_c} \quad \text{for high bond bars.} \tag{8}$$

Table 2: Concrete strength classes, characteristic compressive strengths f_{ck} (cylinder) mean tensile strength f_{ctm}, and characteristic tensile strengths f_{ctk} of the concrete (in N/mm²). (The classification of concrete eg, C20/25 refers to cylinder/cube strength as defined in Section 7.3.1.1 of ENV 206).

Strength Class of Concrete	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck}	12	16	20	25	30	35	40	45	50
f_{ctm}	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1
$f_{ctk,0.05}$	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9
$f_{ctk,0.95}$	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3

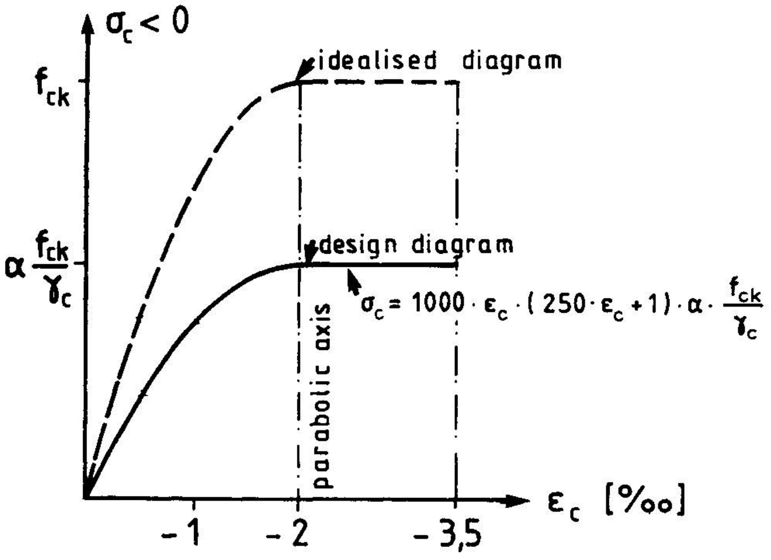

Figure 1: Parabolic-rectangular stress-strain diagram for concrete in compression

2.3 Deformation Properties

The values of the deformation properties depend not only upon the concrete strength class but also upon other parameters such as the properties of the aggregates, the mix design, the environment and the conditions of use in general.

Where an accurate calculation is considered necessary, they should be established from known data appropriate to the particular conditions. Nevertheless, for many calculations an appropriate estimate will usually be sufficient and data for *instantaneous* and *time dependent* deformations of concrete for those cases are given in Para 3.1.2.5 and 4.2.1.3 in EC2.

The general form of the *stress-strain* diagram for uniaxial compression is shown schematically in Figure 3.1 of EC2. Idealized stress-strain diagrams for design calculations are given in 4.2.1.3.3: for structural analysis the idealization is given by a mathematical model expressed by the function (4.2), for cross-section design the preferred idealization is the parabolic rectangular diagram which is shown in Figure 1 of this paper (identical to Figure 4.2 of EC2).

Values of the secant modulus of elasticity E_{cm} are given in Table 3.2 of EC2.

Poisson's ratio for elastic strain may be taken equal to 0,2, if cracking is permitted for concrete in tension it may be assumed to zero.

Where thermal expansion is not of great influence, the *coefficient of thermal expansion* may be taken equal to $10*10^{-6}/°C$.

For *time dependent* deformations the data are given in EC2-

Table 3.3 - Final creep coefficient $\phi\ (\infty, t_0)$
Table 3.4 - Final shrinkage strains $\varepsilon_{cs\infty}$.

2.4 Lightweight aggregate concrete

The properties and the technology of lightweight aggregate concrete are given in ENV 206 in addition to those of normal weight concrete.

Lightweight aggregate concrete, denoted by the symbol LC, is classified according to its density in Table 9 of ENV 206, which distinguishes density classes from 1,0 (density 901 to 1000 kg/m^3), 1,2 (density 1001 to 1200 kg/m^3), ... up to class 2,0 (density 1801 to 2000 kg/m^3). All data different from those in Eurocode 2 Part 1 for normal weight concrete are given in Part 1C which is complementary to Part 1 for the use of lightweight aggregate concrete and drafted according to same table of contents.

3 REINFORCING STEEL

Section 3.2 - Reinforcing steel - of Eurocode 2 applies to bars, coiled rods and welded fabrics.

The products are classified according to grade (denoting f_{yk}), class (indicating the ductility characteristics), size, surface characteristics and weldability.

Two shapes of *surface characteristics* are defined:

- ribbed bars, indicated by the value of the projected rib factor f_{Rk}, resulting in high bond action when the characteristic value f_{Rk} is not less than that specified in EN 10 080;
- Plain smooth bars, resulting in low bond action (ribbed bars not satisfying the requirements for f_{Rk} should be treated as plain bars with respect to bond).

Two *classes of ductility* are defined:

- High (H): $\varepsilon_{uk} > 5{,}0$ %; value of $(f_t/f_y)_k > 1{,}08$ (9)
- Normal (N): $\varepsilon_{uk} > 2{,}5$ %; value of $(f_t/f_y)_k > 1{,}05$

It is likely that, during the ENV period, a higher ductility steel (class S) will be introduced for use in seismic regions.

For *structural analysis* in ultimate limit states, the plastic approach may be used only for very ductile structural elements where high ductility steel is used (Para 2.5.3.2.2). Using linear analysis with redistribution, the condition related to the steel, allowing the omitance of an explicit check on the rotation capacity of critical zones is

$\delta \geq 0{,}7$ for class H and $\delta \geq 0{,}85$ for class N, (10)

δ being the ratio of the redistributed moment to the moment before redistribution (para 2.5.3.4.2).

For overall analysis, an idealized bi-linear *stress-strain diagram* may generally be used. It is given as Figure 2 of this paper (identical to Figure 4.5 of EC2). It may be modified, e.g. with a flatter or horizontal top branch, for local verifications and section design.

prENV 10 080 gives the methods of production, the specified characteristics, the methods of testing and the methods of attestation of conformity for reinforcing steel.

It considers weldable steel, specifies f_{Rk} in function of the diameter (Table 5 of prEN 10 080), considers one grade $f_{yk} \geq 500$ N/mm², and two ductility classes H and N, denominated B 500 H and B 500 N in Table 3 of this paper (identical to Table 1 of prEN 10 080).

Table 3: Properties of reinforcing steel grades B 500 H and B 500 N of various product forms

1	Product form	Bars		Coils		Welded Fabric		p[1)]
2	Steel grade	B 500 H	B 500 N[2)]	B 500 H	B 500 N	B 500 H	B 500 N	
3	Nominal size [3)] (mm)	6 to 40	6 to 16	6 to 16	4 to 16	6 to 16	4 to 16	-
4	Yield strength R_e (N/mm²)	500	500	500	500	500	500	0,95
5	Ratio R_m/R_e*)	1,08	1,03*) or 1,05	1,08	1,03*) or 1,05	1,08	1,03*) or 1,05	0,95*)
6	Total elongation *) (%) at max. force A_{gt}	5,0	2,0 *) or 2,5	5,0	2,0*) or 2,5	5,0	2,0 *) or 2,5	0,95*)
7	Suitability for bending (Rebend test)	Table 3	Table 3	Table 3	Table 3	Table 3	Table 3	
8	Fatigue strength (N/mm²) (stress range $2\sigma_A$)	200 4)	200	200	200	100	100	*) 5)
9	Strength of welded joints (N)	-	-	-	-	$0,3.R_e$xA 6)	$0,3.R_e$xA 6)	0,95
10	Permissible deviation from nominal mass (%)	-4,5	-4,5	-4,5	-4,5	-4,5	-4,5	0,95
11	Projected rib area f_R	Table 5	Table 5	Table 5	Table 5	Table 5	Table 5	7)
12	Chemical composition and carbon equivalent	Table 2	Table 2	Table 2	Table 2	Table 2	Table 2	8)

1) See 3.6 (1-α=0,90 in all cases).
2) Cut from coil and straightened.
3) Details see Table 4.
4) 150 N/mm² for nominal sizes > 20 mm.
5) Type test, see note to 5.5.4.
6) A: Nominal cross sectional area of the thicker wire.
7) Minimum values.
8) Maximum values.
*) Values and/or their means of evaluation under discussion.

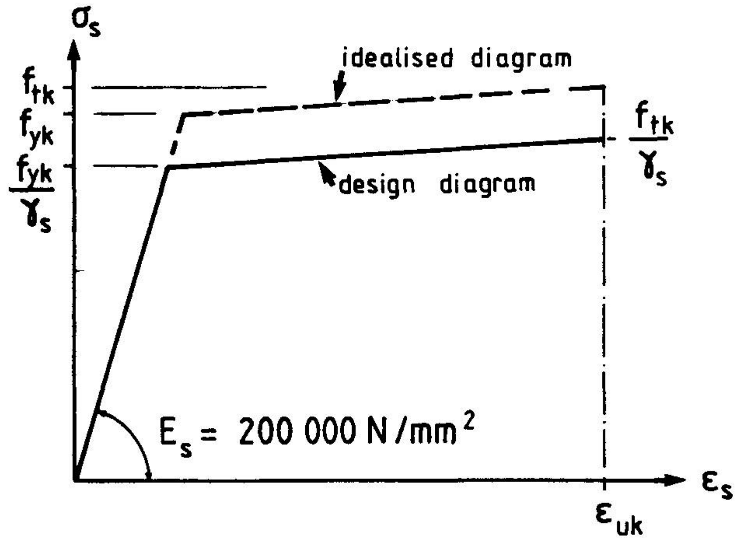

Figure 2: Design stress-strain diagram for reinforcing steel

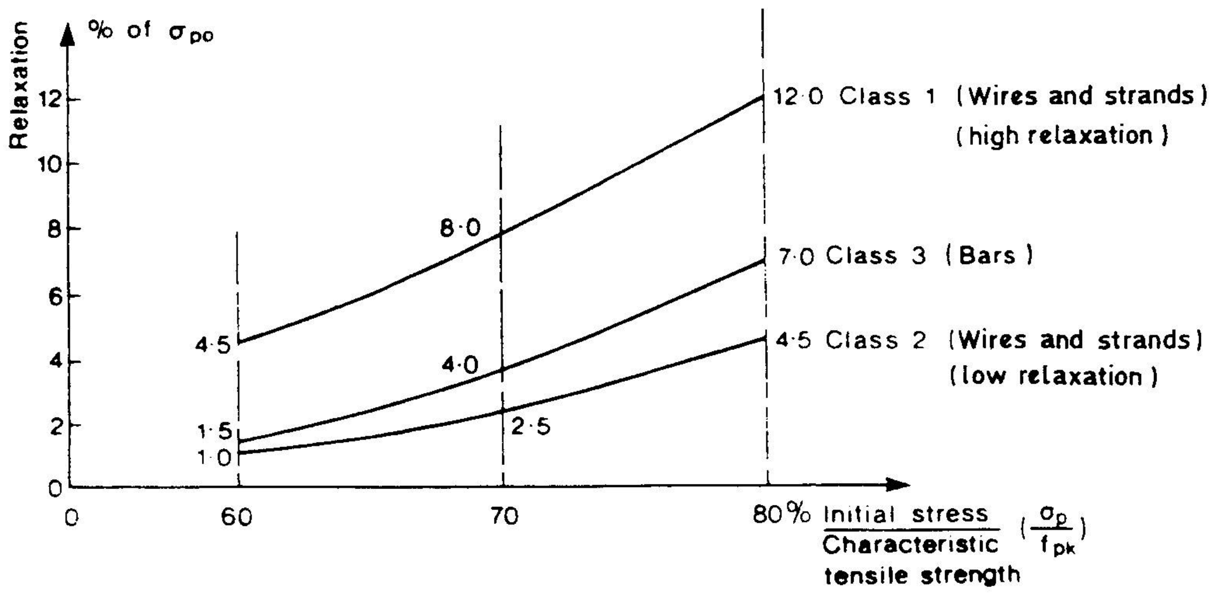

Figure 3: Relaxation losses after 1000 h at 20° C for relaxation classes

1: Wires, strands with high relaxation
2: Wires, strands with low relaxation
3: Prestressing Bars

4 PRESTRESSING STEEL

Section 3.3 "Prestressing Steel" of Eurocode 2 applies to wires, bars and strands used as prestressing tendons.

The products are classified according to grade ($f_{p0,1k}$ and f_{pk}), class (indicating the relaxation behaviour), size and surface characteristics.

Three *classes of relaxation* are defined (Figure 3):

Class 1: for wires and strands, high relaxation
Class 2: for wires and strands, low relaxation
Class 3: for bars.

According prEN 10 138-1, curves for relaxation of load shall be established, at a nominal temperature of 20° C, for a period of 1.000 h from an initial load of 70 % of the actual breaking load.

According 4.2.3.4.1 of Eurocode 2 the long term values of the relaxation losses may be assumed to be three times the relaxation losses after 1000 h; Fig. 3 gives relaxation losses for other values of the initial stress, needed for design purposes.

prEN 10 138-2 and 3 specify low relaxation, 2,5 %, respectively for all stress relieved cold drawn wire and strand.

Strength is specified by the characteristic breaking load, the characteristic 0,1% proof load and the maximum load (which is 1,15 * characteristic breaking load for wire and strand according prEN 10 138-2 and 3).

The products shall have adequate *ductility* in elongation and bending (para 3.3.4.3). Adequate ductility is assumed by specified minimum elongation at maximum load (3,5 % for wire and strand according prEN 10 138-2 and 3) and by requirements for bendability in reverse bends testing for wire and constriction at break for wire and strand.

Stress-strain diagrams for the products shall be prepared and made available by the producer as an annex to the certificate accompanying the consignment. Such as for reinforcement steel, an idealized bi-linear diagram (Figure 4.6 of EC2) may generally be used for design purposes.

The products shall have adequate *fatigue strength*. According prEN 10 138-1 the material shall withstand without failure two million cycles of stress fluctuation down from a maximum stress of 70 % of the actual strength. The fluctuating stress range is 200 N/mm² for smooth wires, 180 N/mm² for indented wires, 190 N/mm² for smooth wire strands and 170 N/mm² of indented wire strands.

Dimensions and properties of stress relieved cold drawn wires are given in Table 4 (identical to Table 2 of prEN 10 138-2) and those of strands in Table 1 of prEN 10 138-3.

Table 4: Dimensions and properties of stress relieved cold drawn wire

Nominal (1)				Specified									
Diameter d	Tensile strength (2)	Cross sectional area	Mass (3)	Tolerance on cross sectional area	Characteristic breaking load F_m	Characteristic 0.1% proof load (4) $F_{p0.1}$	Maximum breaking load (5)	Minimum elongation at max load $L_0 \geq 200$ min Agt	Constriction at break	Minimum number of reverse bends	Bend radius for reverse bends	Max relaxation at 1000 h (6)	Fatigue stress range
mm	N/mm²	mm²	g/m	+/-mm²	kN	kN	kN	%			mm	%	N/mm²
4.0	1770	12.6	98.9	0.25	22.3	19.2	25.6	) for	) for	) 4 for	10	) for	) 200
4.0	1860	12.6	98.9	0.25	23.4	20.1	26.9	) all	) all	) smooth	10	) all	) for
5.0	1670	19.6	154	0.39	32.7	28.1	37.6	) wires	) wires	) wires	15	) wires	) smooth
5.0	1770	19.6	154	0.39	34.7	29.8	39.9	) 3.5 %	) a ductile	)	15	) 2.5 %	) wires
6.0	1670	28.3	222	0.47	47.3	40.7	54.4	)	) break	)	15	)	)
6.0	1770	28.3	222	0.47	50.1	43.1	57.6	)	) visible	) 3 for	15	)	) 180
7.0	1670	38.5	302	0.58	64.3	55.3	73.9	)	) to the	) indented	20	)	) for
7.5	1670	44.2	347	0.66	73.8	63.5	84.9	)	) unaided	) wires	20	)	) indented
8.0	1670	50.3	395	0.75	84.0	72.2	96.6	)	) eye	)	20	)	) wires
9.4	1570	69.4	545	1.00	109.0	90.5	125.4	)	)	)	25	)	)
10.0	1570	78.5	616	1.10	123.0	102.0	141.5	)	)	)	25	)	)

Note (1) The nominal modulus of elasticity may be taken as 205 kN/mm².

Note (2) The nominal tensile strength is calculated from the nominal cross-section and the specified characteristic breaking load.

Note (3) The mass is calculated from the nominal cross-section area and a density value of 7.85 kg/dm^3.

Note (4) For wires larger than 8 mm the specified characteristic proof load is approximately 83 % of the specified characteristic breaking load. For wires of 8 mm and smaller the corresponding figure is 86 %.

Note (5) The maximum breaking load is 1.15 * specified characteristic breaking load.

Note (6) For nominal diameters 9.4 mm and 7.5 mm the relaxation requirement may be varied by agreement between supplier and purchaser.

The procedure of attestation of conformity by *certification* is published as Annex A of prEN 10 138-1 for detailed comments and will be covered by a separate document.

5 PRESTRESSING DEVICES

Chapter 3.4 of EC2 summarizes only the basic requirements for anchorages and couplers, ducts and sheath and for the design of anchorage zones. For the quantification of these requirements, reference is made to relevant (future) Standards and approval documents.

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EC 2: Structural Analysis

EC 2: Analyse des structures

EC 2: Schnittgrössenermittlung

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SUMMARY

Relevant chapters of EC 2 Part 1 dealing with the structural analysis of concrete structures will be presented and discussed. Particular emphasis is laid on non-linear and plastic methods of analysis, the check of rotation capacity as well as on simplified approaches based on non-linear material behaviour. The presentation covers both the serviceability limit states and ultimate limit states.

RESUME

Les chapitres de la Partie 1 de l'EC 2 qui se rapportent à l'analyse des structures en béton sont présentés et discutés. En particulier, l'importance est attribuée au comportement non-linéaire et plastique, à la vérification de la rotation plastique admissible des sections ainsi qu'aux méthodes d'analyse simplifiées qui sont basées sur un comportement non-linéaire de la structure. En outre, les états-limites ultimes et les états-limites de service sont considérés.

ZUSAMMENFASSUNG

Die einschlägigen Kapitel des EC 2 Teil 1 bezüglich der Schnittgrössenermittlung in Betontragwerken werden angesprochen und erörtert. Der Schwerpunkt liegt hierbei auf nichtlinearen und plastischen Verfahren, der Kontrolle der Rotationsfähigkeit von Querschnitten sowie auf vereinfachten Verfahren auf der Grundlage eines nichtlinearen Materialverhaltens. Die beiden Nachweisbereiche Gebrauchszustand und Versagenszustand werden behandelt.

1 STRUCTURAL ANALYSIS IN EUROCODE 2

For structural analysis four methods are available in EC 2 excluding the regulations for second order effects (buckling):

- The Theory of Elasticity
- The Theory of Elasticity with Redistribution
- The Nonlinear Approach
- The Method of Plasticity

The theory of elasticity is well known to everybody and does not need any further comment. As the method of plasticity, as well as the application of the theory of elasticity with redistribution, is a subgroup of the more general nonlinear method, the latter will be treated in detail in the following, discussing also the mentioned subgroup's applications in EC 2. This presentation is a compendium of several other publications [1],[2],[3],[4] giving more details.

2 WHAT DOES NONLINEAR DESIGN MEAN ?

Design according to the **Theory of Elasticity** with characteristic loads and partial safety coefficients for actions means that one first calculates internal forces and moments using an elastic constitutive law A (Fig. 1)

$$\sigma = \epsilon * E \quad or \quad M = - y''/(EJ), \quad N = u' * EA. \tag{1}$$

With the exception of the modulus of elasticity this constitutive relation is material independent and linear. Therefore the principle of superposition holds as long as geometrical nonlinearities - buckling - are also excluded.

γ_B $\boxed{\text{Load}}$ $\downarrow$ Constitutive Law A $\downarrow$ γ_B Moment

$\leqq$

$\boxed{\text{Material}}$ $\downarrow$ Constitutive Law B $/\gamma_M$ $\downarrow$ Moment (γ_M)

Fig. 1 Current Safety Format

Then in a second step the different cross-sections are designed with realistic nonlinear constitutive laws (Fig. 1 and 2) for steel and concrete, regarding characteristic material parameters and partial safety coefficients for resistance.

In the case of **Nonlinear Design** only one set of realistic constitutive laws is applied (B in Fig. 1) the whole process. This means, when deriving the differential equations - neglecting normal forces - for the beam, that one has to substitute the elastic relation (see also Fig. 3)

$$M_i = - y'' / (EJ) \tag{2}$$

by the nonlinear moment-curvature-relation:

$$M_i = f(y''). \tag{3}$$

In this way a complete F-y-diagram can be calculated, increasing the load step by step by numerical means using for example a computer (Fig. 4). Such an investigation starts within the serviceability range accounting also for cracks and realistic deformation and goes on until the computer program reaches a given limiting condition such as e.g. a maximum concrete strain or a limiting steel strain. The so determined load is the bearing capacity of the beam resp. beam system including already redistribution of forces and moments due to the changing stiffness conditions.

Since this method is able to simulate experiments very well, this approach gives the real behaviour and is declared to be the basic method in the CEB-Model Code [5]. Slabs, walls and shells may also be analyzed by this method.

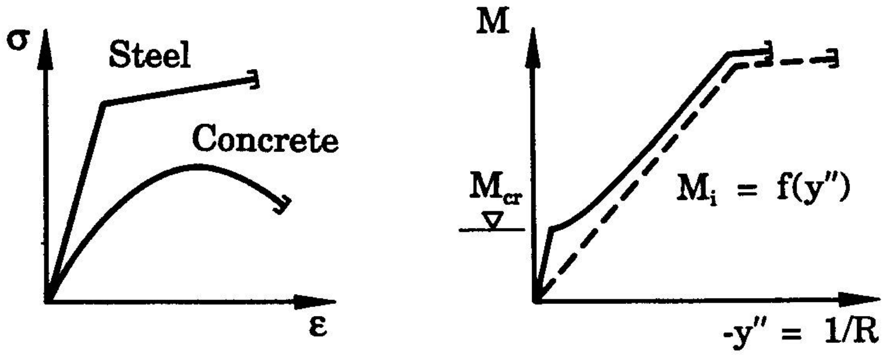

Fig. 2 Constitutive Law and Moment-Curvature-Relation

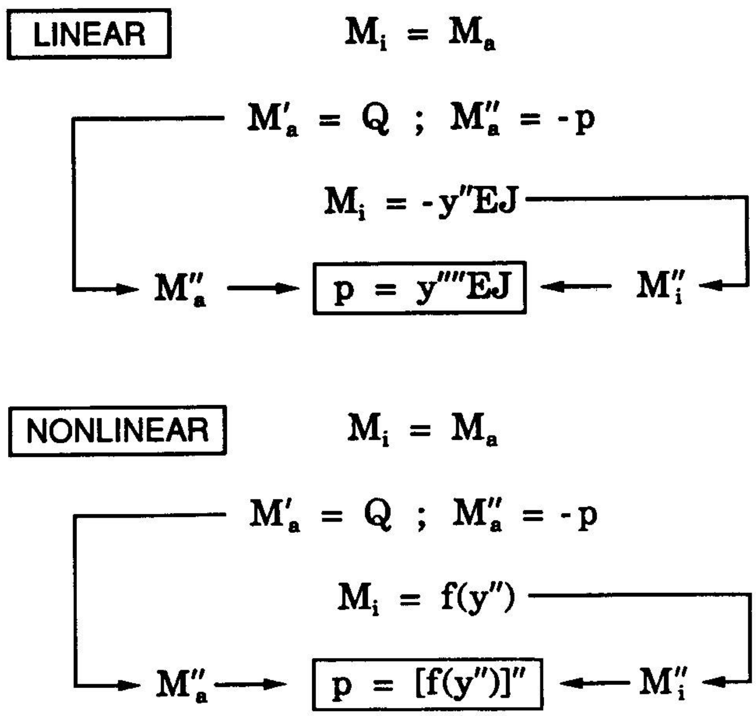

Fig. 3 Linear and Nonlinear Design

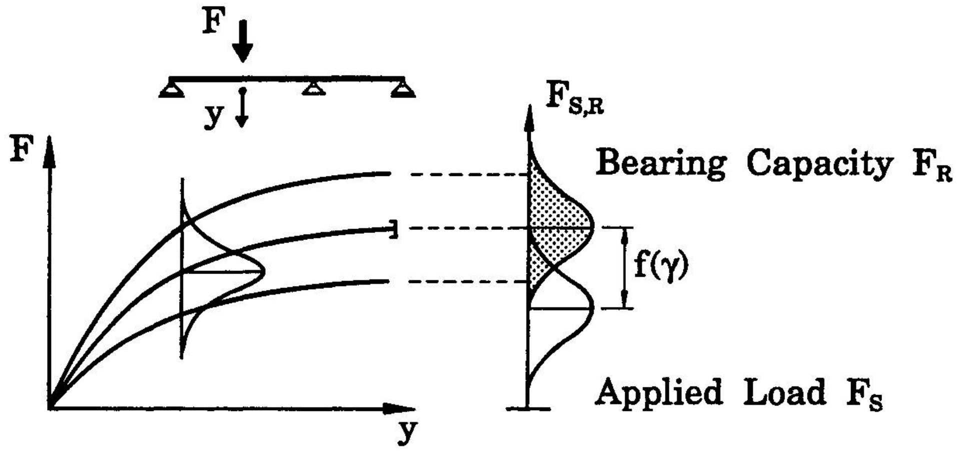

Fig. 4 Proposed Safety Format

With regard to the real behaviour, the **Plastic Method** as addressed in EC 2 is an approximate method derived from this basic method. It allows to concentrate the summed up curvatures along the beam in discrete hinges. This is justified by the fact that big cracks usually appear at special locations of maximum moment and by assuming rigid ranges in between.

It is based on the theory of plasticity with its two limiting theorems for an upper and a lower bound of a system's bearing capacity, stated in engineering terms as:

Theorem 1: A load system which belongs to an admissible stress state, not violating yield conditions, is a lower bound for the bearing capacity of the structure.

Theorem 2: A load system being in equilibrium in a kinematic admissible state of motion is an upper bound for the bearing capacity of the structure.

The so-called **Redistribution of Moments** is an application of this plastic method.

3 ADVANTAGES AND DISADVANTAGES

The theory of elasticity

- Is easy to handle
- Allows superposition to be applied
- Changes of the reinforcement do not influence the internal force and moment distribution
- The serviceability range is rather well approximated
- The ultimate limit state is not covered correctly (Fig. 5).

A complete nonlinear design

- Gives correct results for the serviceability (SLS) as well as for the ultimate limit state (ULS).
- Reinforcement influences the distribution of inner forces and moments.

- The principle of superposition is no longer valid and load combinations have to be considered.
- Demands numerical means and in general a computer.

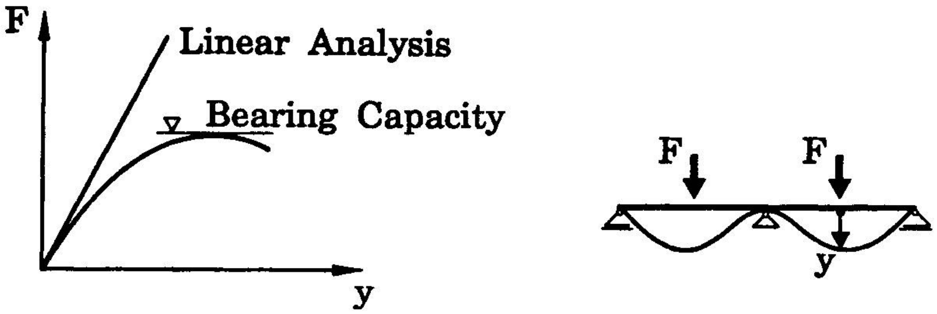

Fig. 5 F-y-Relation for Linear and Nonlinear Design

With regard to the safety format, it has to be mentioned that a safety check on **the Level of Internal Forces and Moments** can never consider system failures correctly, whatever method is used. For example slabs do not fail, when discrete moments caused by loads exceed the design moments. One cross-section failure within a multispan continuous girder may not cause failure of the whole system as it is the case for a similar type failure in a single span girder.

However, with a consistent nonlinear design, where the safety check is done at the level of loads comparing the bearing capacity of the whole structure with the acting loads, automatically regards the different failure behaviour of different systems.

As it is possible to consider the scatter of the F-y-curve (Fig. 4) explicitly, also the scatter of the bearing capacity may be determined. Comparing the density function for the bearing capacity with the density function of the acting loads, the probability of failure of the system may be evaluated and simply controlled.

3 APPLICATION IN EC 2

EC 2 demands the application of the theory of elasticity for the evaluation of internal forces and moments to check the SLS state.

For the ultimate limit state (ULS)

- a design procedure by means of the theory of elasticity and a following cross-section design at the level of internal forces and moments is allowed as well as
- a limited redistribution of elastic moments, if a few requirements are fulfilled garanteeing a minimum of ductility resp. rotation capacity at locations of maximum moments.
- The method of plasticity allows an unlimited redistribution, if in case of beams the rotation is checked and in case of slabs a limit of the relation between support and field moments is regarded. In a recommendation it is further stated that at cases of extreme redistribution only normal ductile steel according to Appendix 2 should be used.

The method of plasticity is very simple and can be applied easily when a calculation by hand is intended. In case of a multispan continuous girder e.g. one just has to assume support moments for loads increased by a partial safety factor and to fulfill the equilibrium conditions between the supports by appropriate field moments before doing a cross-sectional design using characteristic material values and appropriate partial material safety coefficients.

It is a safe method in the sense of theorem 1 of the theory of plasticity as a state of equilibrium is given not violating yield conditions. The so-called Hillerborg strip method for plates belongs to the same class, while the yield line theory according to theorem 2 gives only an upper bound solution, i.e. the real bearing capacity of the plate may be smaller.

Also the use of strut and tie models to investigate walls and in plane loaded plates as well as the so-called D-regions of beams and corbels is a special application of the method of plasticity similar to the Hillerborg strip method. As long as the equlibrium conditions are fullfilled within a properly selected truss system provided with sufficient ductility - according to theorem 1 ''not violating yield conditions'' - a safe solution may be reached.

When applying the method of nonlinear design according to EC 2 also, a primary evaluation of internal forces and moments is required using mean values of material parameters followed by a nonlinear cross-sectional design with partial safety factors and characteristic materials values. This always leads to an oversafe design as the initial assumption of reinforcement on the basis of **mean** values is always increased at the second stage of the cross-sectional design according to partial safety coefficients and **characteristic** material parameters.

The still existent deficiency is the afore mentioned problem that in general a system failure can never be covered realistically for all methods using a cross-sectional design. It will especially underestimate the safety of slabs, walls and shells.

It is more reasonable to compute - nonlinear design by hand is usually not possible - in one single numerical approach the bearing capacity by strain limits, neglecting an evaluation of internal forces and moments and to ensure safety by an appropriate safety concept with slightly changed partial safety coefficients at a comparison level of loads (Fig. 4).

In this case of a consistent nonlinear investigation SLS and ULS may be covered by the **same** method.

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EC 2: Design for Ultimate Limit States

EC 2: Vérification aux états-limites ultimes

EC 2: Bemessung in den Grenzzuständen der Tragfähigkeit

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SUMMARY

This paper outlines the provisions given in Eurocode 2 for the design of elements for the ultimate limit state. The subjects covered are: design for bending with or without axial force, shear, torsion, punching shear and the effects of structural deformations (buckling). For each mode of behaviour the main features of the methods given in the code are described.

RESUME

Cet article présente les recommandations de l'Eurocode 2 pour la vérification des éléments structuraux aux états-limites ultimes, en particulier les états-limites ultimes pour les sollicitations d'effort normal et de flexion, pour les sollicitations d'effort tranchant, de la torsion, du poinçonnement ainsi que les états-limites atteints par déformation structurale (flambement). Pour chacun des états-limites ultimes, les principes les plus importants dans l'Eurocode 2 sont énoncés.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt die Festlegungen in Eurocode 2 für die Bemessung von Bauteilen in den Grenzzuständen der Tragfähigkeit. Im einzelnen werden behandelt: Bemessung für Biegung mit oder ohne Längskraft; Schub, Torsion, Durchstanzen sowie die Auswirkungen von Tragwerksverformungen (Knicken). Für jeden dieser Grenzzustände werden die wesentlichen Nachweisverfahren in EC 2 beschrieben.

1. INTRODUCTION

The ultimate limit states are treated in chapter 4.3 of the Code. This chapter does not, of course, stand alone, but draws particularly on material in Chapter 2 (Partial Safety Factors and Analysis) and chapter 4.2 (Design Material Properties). Satisfactory design for the ultimate limit state also depends upon applying the provisions of Chapter 5, Detailing Provisions.

The basic principles and methods proposed for the treatment of the ultimate limit states follow closely those given in the CEB Model Code of 1978. The CEB proposals have, however, been amended in detail for various reasons. Firstly, simplicity. The CEB Code, being a Model Code, can afford to develop more complex and rigorous methods than can be done in an operational code. Furthermore, operational codes cannot afford to include too many alternative methods of design. Secondly, development of knowledge. In some areas new research has allowed improvement to the CEB proposals. EC2 has attempted to take account of the latest developments wherever possible.

The organisation of the chapter is as follows:

- Bending and longitudinal force	4.3.1
- Shear	4.3.2
- Torsion	4.3.3
- Punching	4.3.4
- Buckling	4.3.5

Each of these subject areas will be covered briefly in the following sections.

2. BENDING AND LONGITUDINAL FORCE

This section follows very closely the proposals in the CEB Model Code. The design stress-strain curves for ordinary reinforcement and concrete are shown in Figure 1(a) and (b). It should be noted that, for both, possible alternatives are suggested. Figure 1(c) indicates the assumptions relating to the strain distribution at ultimate for reinforced concrete. For prestressed sections, allowance has to be made in assessing the steel strain for the prestrain in the tendons. The indicative (boxed) values given in EC2 for the partial safety factors on the steel and concrete strengths are, respectively, 1.15 and 1.5.

3. SHEAR

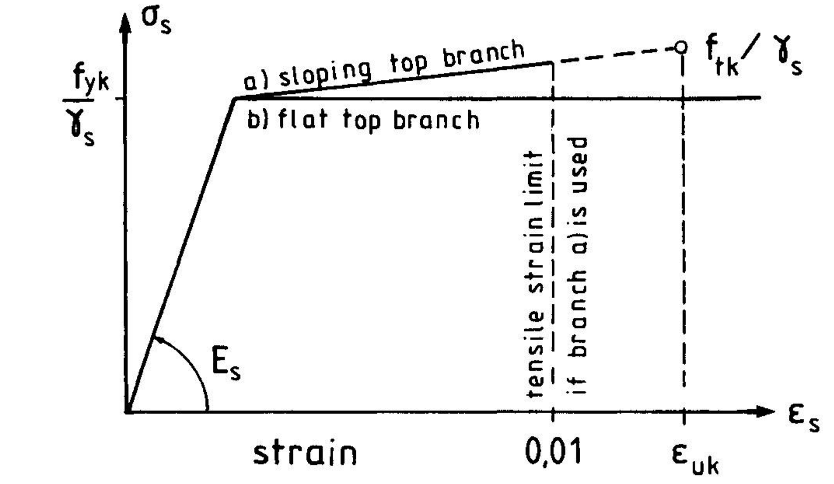
There are three basic values defined for shear resistance. These are:

- V_{Rd1} - the design shear resistance of the member without shear reinforcement
- V_{Rd2} - the maximum design shear force that can be carried without crushing of the notional concrete compressive struts
- V_{Rd3} - the design shear force that can be carried by a member with shear reinforcement

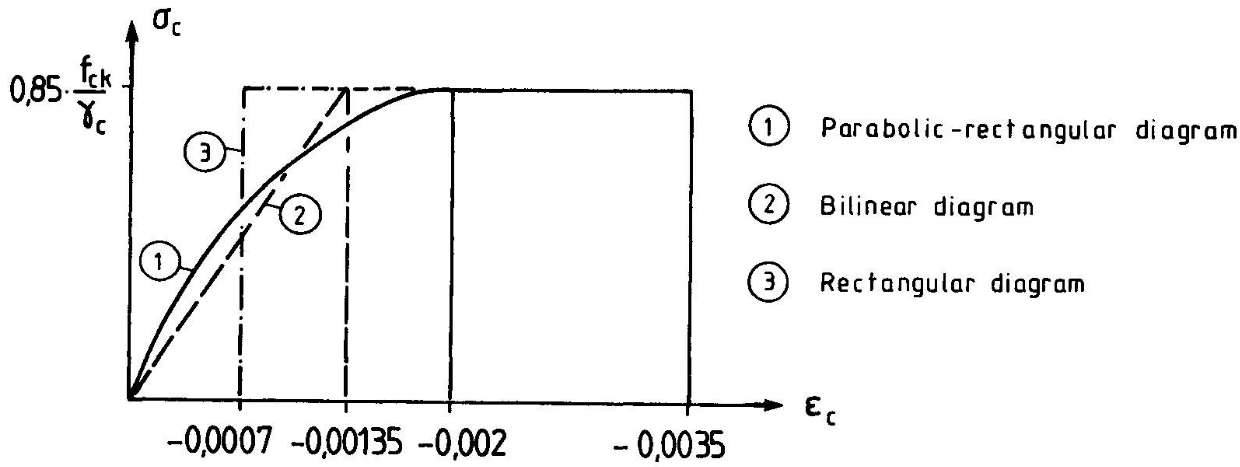
If the design shear, V_{Sd}, is less than V_{Rd1}, only a minimum amount of shear reinforcement need be provided. This minimum may generally be omitted in slabs and members of minor importance.

If V_{Sd} exceeds V_{Rd1}, but is less than V_{Rd2}, then shear reinforcement should be provided so that $V_{Rd3} = V_{Sd}$.

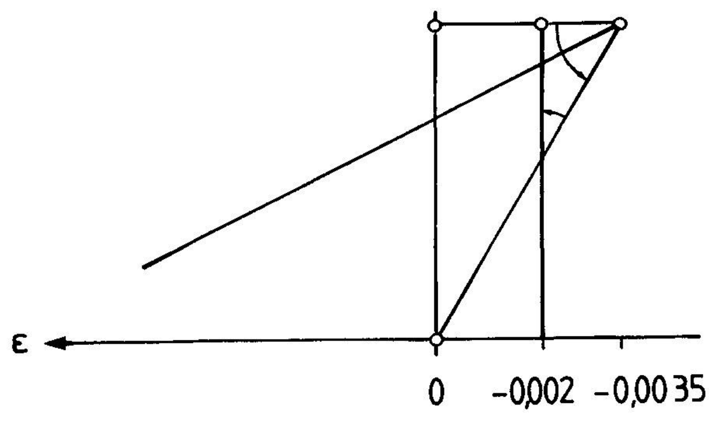
V_{Rd1} is calculated from an empirical relationship which gives the design stress as a function of the tensile strength of the concrete, the reinforcement ratio, the average longitudinal stress, and, for members less than 600 mm deep, the section depth. V_{Rd1} may also be adjusted to allow for enhanced strength close to supports. This relationship has been justified against a very large population of test data.



(a) Design stress-strain curves for reinforcement



(b) Concrete design stress-strain curves



(c) Ultimate strain profiles

Fig. 1 Assumptions for design for flexure

Two methods are given for assessing V_{Rd2} and V_{Rd3}. Both are based on the assumption of a notional truss within the beam where the tension members are formed by the flexural tension reinforcement and the shear reinforcement, while the compressive forces are carried by the concrete in the compression zone and by notional struts within the concrete (see Figure 2).

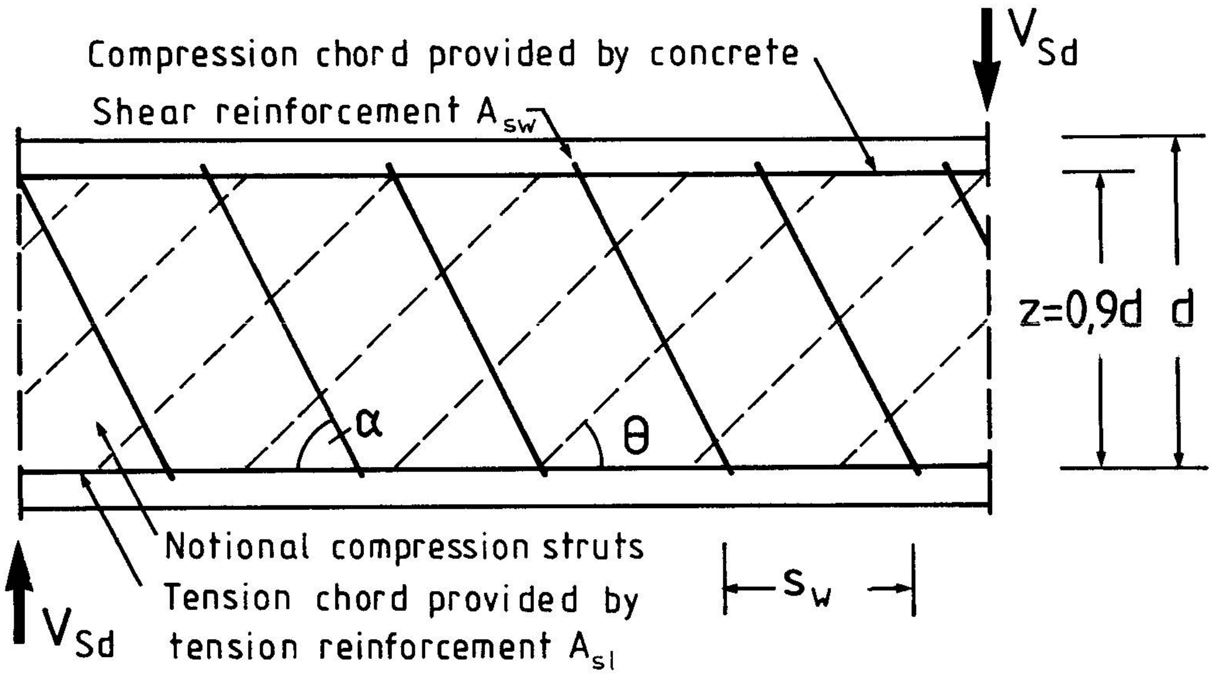


Fig. 2 Assumptions for the calculation of shear reinforcement

In the 'Standard Method', the struts are assumed to be aligned at an angle, θ, of 45° to the axis of the beam and reinforcement is only required to carry the excess shear force above V_{Rd1}. This gives, for vertical stirrups:

$$V_{Sd} = V_{Rd3} = V_{Rd1} + 0.9\ d\ f_{ywd}\ A_{sw}/s \qquad (1)$$

and $$V_{Rd2} = 0.45 \upsilon b_w d\ f_{cd} \qquad (2)$$

where
- d is the effective depth
- f_{ywd} is the design strength of the shear reinforcement
- A_{sw} is the cross-sectioned area of the shear reinforcement
- s is the spacing of the shear reinforcement
- b_w is the minimum web breadth
- f_{cd} is the design strength of the concrete
- υ is an empirical effectiveness factor varying from 0.5 to 0.6 over the practical range of concrete strengths

In the 'variable strut inclination method', the angle θ in Figure 2 may be selected by the designer within a range which can be as great as $0.4 < \cot\theta < 2.5$. Once the design shear exceeds V_{Rd1}, all the shear force has to be carried by shear reinforcement. For vertical stirrups, this gives the following relationships for V_{Rd2} and V_{Rd3}:

$$V_{Rd3} = 0.9\,d\,f_{ywd}A_{swc}\cot\theta/s \qquad (3)$$

$$V_{Rd2} = 0.9 b_w d \nu f_{cd}/(\cot\theta + \tan\theta) \qquad (4)$$

A possible design procedure is to take either the maximum permitted value of cot θ or, if less, the value of cot θ which gives $V_{Sd} = V_{Rd2}$ and calculate the amount of shear reinforcement on the basis of this value. It should be noted that the choice of cot θ will influence the curtailment of reinforcement.

4. TORSION

The approach adopted for design for torsion is an extension of the 'variable strut inclination method' described above. Two torsional resistances are defined:

- T_{Rd1} - the maximum torsion that can be resisted by the compressive struts in the concrete (torsional equivalent of V_{Rd2})
- T_{Rd2} - the maximum torsion that can be resisted by the reinforcement (torsional equivalent of V_{Rd3})

Both these quantities are a function of the strut angle, θ and, where combined shear and torsion are considered, the same angle must be chosen for both calculations.

Rules are given for the design of combined shear and torsion or torsion combined with bending and/or axial force. Conditions are also set out for cases where only a minimum area of stirrups is required.

5. PUNCHING

Punching may also be considered as an extension of the shear provisions. A critical perimeter around a column is defined as shown in Figure 3 and the design shear force is assessed for this perimeter.

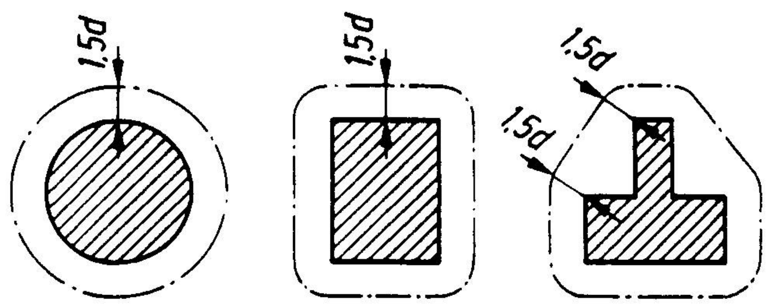


Fig. 3 Perimeters for punching

From this a shear per unit length of the perimeter, v_{Sd}, is calculated from the relation:

$$v_{Sd} = V_{Sd}\, \beta / u \tag{5}$$

where u is the length of the perimeter
β is a coefficient which takes account of the effects of eccentricity of loading (moment transfer between column and slab)

If v_{Sd} is less than the design shear resistance per unit length of the perimeter for the slab without shear reinforcement, v_{Rd1}, then no shear reinforcement is required. For greater shears, shear reinforcement is required. Shears in excess of 1.6 v_{Rd1} cannot be supported. The expressions for v_{Rd1} and v_{Rd3}, the shear capacity of the slab with shear reinforcement, are effectively the same as for ordinary shear.

There is also a requirement for a minimum design moment in the region of the slab-column connection.

6. ULTIMATE LIMIT STATE INDUCED BY STRUCTURAL DEFORMATION (BUCKLING)

The design procedure envisaged for dealing with slenderness effects is, briefly, as follows:

(i) The structure is classified as:

(a) braced or unbraced

and (b) sway or non-sway

A braced structure is one where all horizontal loads may be assumed to be carried by stiff, bracing elements such as walls.

A sway structure is one where the deflection of the connections has a significant effect on the bending moments.

(ii) Depending on the classification, the vertical members are checked to establish whether they are slender. The effects of deflection may be ignored in non-slender members but must be taken into account in slender members.

(iii) Where necessary, the members are designed to take account of the effects of the deflections.

In non-sway structures, the individual columns are treated as isolated columns which may be assumed to deflect as shown in Figure 4(a). In sway structures, the whole structure will deflect as shown in Figure 4(b). In addition to considering sway of the whole structure, however, it is also necessary to consider the possibility of each column individually deflecting as in Figure 4(a).

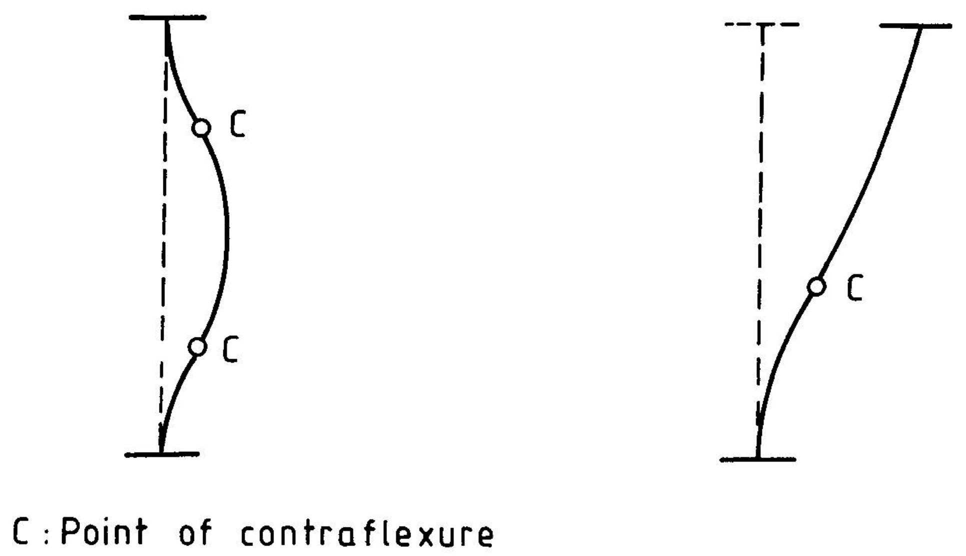
The code only develops a simplified design method for isolated columns. For other situations a more rigorous method is needed and the necessary assumptions for this are set out in Appendix 3. The procedure for isolated braced columns is as follows:

(i) The slenderness ratio $\lambda = l_o/i$ is calculated. l_o is the effective length of the column and i the radius of gyration of the section.

(ii) If $\lambda < 25$, the structure is not slender.

(iii) If $25 < \lambda < 25(2 - e_{o1}/e_{o2})$ then it is only necessary to ensure that the ends of the column can withstand a moment greater than $N_{Sd}\, h/20$. e_{o1} and e_{o2} are, respectively, the numerically smaller and larger end eccentricities.

(iv) If $\lambda > 25(2 - e_{o1}/e_{o2})$ then specific measures have to be taken. A simplified method is given for doing this. This is the 'Model Column Method'. The method makes an estimate of the maximum curvature in the column under ultimate conditions and hence an approximate value for the ultimate deflection.



(a) Assumed deflected shape of an isolated braced column

(b) Assumed deflected shape of a column in a sway structure

Fig. 4 Assumed modes of deflection of columns

The column is then designed to withstand the design vertical load, N_{Sd}, acting at an eccentricity e_{tot}, given by:

$$e_{tot} = e_o + e_a + e_2 \tag{6}$$

where e_o is the initial eccentricity estimated from first order analysis. The value chosen is one appropriate to roughly mid-height of the column

e_a is an accidental eccentricity. It is a nominal figure to allow for possible 'out of plumb' construction of the column

e_2 is the ultimate deflection.

Clearly it will frequently be necessary to consider the possibility of the column deflecting about either axis.

Rules are given for deciding whether or not it is necessary to consider bi-axial bending.

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EC 2: Serviceability and Durability

EC 2: Aptitude au service et durabilité

EC 2: Gebrauchstauglichkeit und Dauerhaftigkeit

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SUMMARY
The prescriptions of Eurocode 2 are intended to ensure the required long-term performance. However, the user must apply the relevant prescriptions together with an understanding of engineering models combining the effects of environmental aggressivity, structural form, materials composition, workmanship, and maintenance. Gross error problems cause most premature damage, and improvements at this level will require intensified information and education of persons involved, and specific quality assurance procedures must be enforced.

RESUME
Cette partie de l'Eurocode 2 a été écrite en fonction du comportement à long terme souhaité. Toutefois, l'utilisateur doit non seulement appliquer les prescriptions appropriées du code, il doit aussi tenir compte de l'environnement, de la structure, des matériaux, de la main d'œuvre et de la maintenance. Des erreurs flagrantes à ce niveau sont à la base de la plupart des dommages prématurés. Il faut, pour améliorer la situation, un meilleur réseau d'information, une formation appropriée des utilisateurs et des procédures plus strictes sur le contrôle de la qualité.

ZUSAMMENFASSUNG
Die Vorschriften in EC 2 sollen die erforderliche Lebensdauer gewährleisten. Allerdings muss sie der Benützer mit Verständnis für ingenieurmässige Modellbildung anwenden, das aggressive Umwelteinwirkungen, Formgebung, Ausführungsqualität und Unterhaltung umfasst. Vorzeitige Schäden sind zumeist auf grobe Fehler zurückzuführen, deren Vermeidung intensive Information und Weiterbildung der Beteiligten, aber auch die Durchsetzung spezieller Qualitätssicherungsverfahren verlangen wird.

Background

Concrete has for many years been believed unconditionally to be a very strong and robust material and in its own right to "be strong and durable as rock". Based on today's knowledge of the material characteristics of concrete and with the past two decades of experience with the performance of concrete structures in aggressive environments, a much more differentiated judgement must be made.

It has become painfully clear that concrete is not a foolproof material although its fundamental ingredients are available in abundance and its manufacture requires no special skills,-apparently. The simplicity of old times concrete, and its low cost compared to other available building materials, has made concrete and reinforced concrete the most used building material in the world, appart maybe from sun-dried clay. The availability of such a low cost material has been a very large asset to society, but unfortunately a growing discreppancy has developed over the years between the application of concrete in practice and the refinement of materials research and development of modern days concrete. In combination with more and more advanced applications and more and more aggressive environments not fully identified, concrete has in many cases deteriorated at an unacceptable rate.

Recent years strong efforts to take in the lost land has greatly improved our knowledge of how to design, construct and maintain our concrete structures so their original good reputation can be regained.

For the industrialized community this manifests itself through many different channels spanning from awareness of the need to maintain concrete structures regularly, via concious design and construction procedures, to improved education and training of engineers as well as of concrete workers. Governing major parts of this industry is the national and regional codes, standards and specifications. They form a combined technical and legal basis for the building and construction industry, and have thus a tremendous impact on the final outcome from the building industry, thus determining the future performance of our structures.

On the European level the future Eurocodes will govern the construction industry, and Eurocode 2, EC 2, will cover **"Concrete, Reinforced Concrete and Prestressed Concrete Structures"**. Until the end of 1994 this document will be available as a Prestandard open for discussion until the end of 1993. Then the fate of this EC 2 will be determined by the 18 CEN member Countries. Special provisions ensuring durability and long term performance are covered by the European Standard EN 206 on **"Concrete. Performance, production, placing and compliance criteria"**. Currently this standard is available as a Prestandard, ENV 206, also open for discussion, and changes in the current text may be expected before the final version is obtained.

Objectives of EC 2 and ENV 206

Within the topics of Serviceability and Durability EC 2 together with ENV 206 shall ensure that concrete structures are designed and constructed so they maintain their required durability and performance for a sufficiently long period of time, which is expected to be in excess of 50 years. ENV 206 itself gives technical requirements for the constituent materials of structural concrete, the concrete composition, the properties of fresh and hardened concrete and their verification. It also covers the production of concrete, its transport, delivery, placing and curing, and the quality control procedures. The standard also ensures that the concretes can be used with the relevant Eurocodes.

Limitations

EC 2 and ENV 206 cover all ordinary type structures with foreseeable environmental conditions and expected normal service lives. However, there are a number of situations where additional, or sometimes even different, requirements may be necessary. This could be for:

- complex structures such as special viaducts, large dams, pressure vessels for nuclear power stations, offshore structures, and for roads

- using new constituent materials, special technologies (e.g. manufacturing processes) or innovating technologies in the building process.

In all such cases the measures chosen shall be suitable and shall not conflict with the requirements for safety and durability of the structure.

Multidisciplinary Problems

Codes and Standards are not foolproof, and following then blindfolded will not necessarily result in satisfactory structures. One of the most important realizations from recent years experience gained with structures in service is, that only by a coordinated effort by all parties involved in all phases of the planning, construction and use of structures can durability problems be avoided throughout the expected lifetime, regardless of how strictly the code or the standard is followed.

This requires cooperation between the following four parties:

- The owner, by defining his present and foreseen future demands and wishes, if any.

- The designers (engineers and architects); by preparing design specifications (including proposed quality control schemes) and conditions.

- The contractor who should follow these intentions in his construction works. Most commonly also subcontractors are involved.

- The user, when he is responsible for the maintenance of the structure during the period of use.

Any of these four parties may - by their actions or lack of actions - contribute to an unsatisfactory state of durability of the structure and thus cause a reduction of the service life. Also interactions between two parties may cause faults which can have an adverse effect on durability and service life.

Modern Durability Technology

Consistent engineering models describing the deterioration mechanisms threatening concrete structures incorporate knowledge from a very wide range of technical disciplines, such as

- statics (structural behaviour)
- materials technology (materials composition)
- design (codes, structural form, design traditions)
- execution (workmanship, local traditions)
- statistics
- economy.

Experience from inspection, maintenance and repair of existing structures must be used to identify and calibrate the critical parameters governing these engineering models.

Based on these models, durability performance can be developed to include the whole range of structural engineering problems from operation, maintenance, inspection, assessment, repair and re-design of existing structures to design and execution of new structures.

Deterioration Mechanisms and Governing Parameters

The number of really significant deterioration mechanisms are few, i.e. only the following four are really important:

- Reinforcement corrosion
- Alkali-aggregate reactions
- Chemical attacks (e.g. sulphate)
- Freeze-thaw bursting

Corrosion destroys primarily the reinforcement and subsequently cracks and spalls the concrete. The three others destroy primarily the concrete.

The presence of water and salt are the two most decisive parameters governing these mechanisms.

Water

All the major deterioration mechanisms require the presence of water in sufficient amounts. Only temperature conditioned cracking, shrinkage cracking, and mechanical wear can take place without water, and such crack formations do not necessarily represent deterioration as such, but can open the concrete for ingress of harmful materials.

Any kind of dry-keeping of the concrete will reduce the rate of development of damage. Indoor concrete is normally sufficiently dry for damage not to develop under normal usages, even if all other conditions for the development of damage are present.

Salt

Chloride based salts are some of the most harmful materials to which concrete can be exposed, either when accidentally mixed into the fresh concrete or when coming on to the concrete surface. The harmful effect is fourfold:

- Chloride based salts provide serious risks for local pit corrosion of the bars when the chlorides reach the reinforcement. This is the most serious threat to concrete structures in the nineties,- as it was in the eighties!

- If the salt contains alkali-metal ions (Na^+, K^+), they also enter the concrete with added risk of alkali-aggregate reactions in case the concrete at the same time contains reactive particles.

- As de-icing agent, salting causes a freeze chock of the concrete surfaces when ice is forced to melt. This can result in thermo-cracks which open the concrete for subsequent ingress of water, salts or other materials. Lamination, spalling and crumbling of the concrete can occur due to combined salting and frost-thaw impacts.

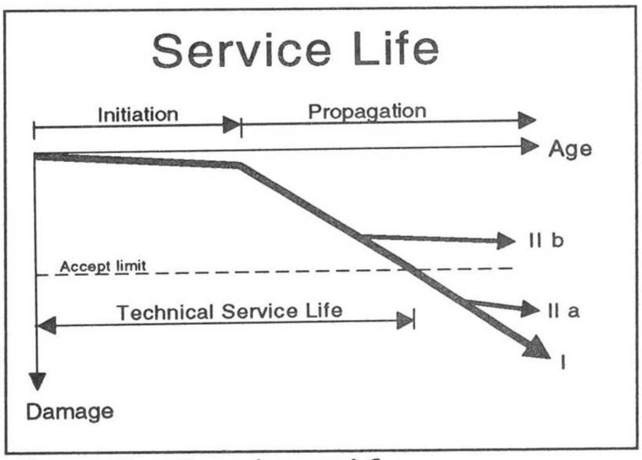
- Salt is hygroscopic since it is retaining water. With salt in the concrete drying out is more difficult and so is stopping possible development of deterioration.

Time Development

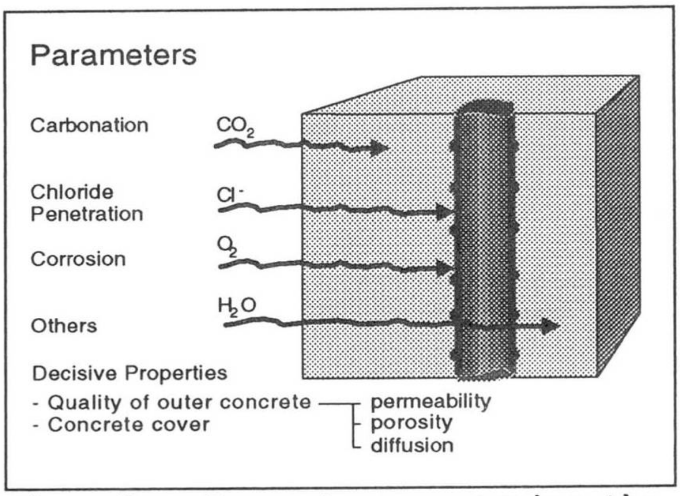
In overall terms nearly all deterioration mechanisms develop in time following a two-phase broken curve as illustrated on Fig.1. The two phases represent:

- An ***initiation phase***, during which no noticeable weakening of the material or of the function of the structure occurs, but some protective barrier is broken down or overcome by the aggressive media. Carbonation, chloride penetration and sulphate accumulation, the latter two accelerated by cyclic wetting and drying, are examples of such mechanisms determining the duration of the initiation period.

- A ***propagation phase***, during which active deterioration normally proceeds rapidly and in a number of cases at accelerating pace. Reinforcement corrosion is one such important example of propagating deterioration.



Figur 1: Service Life



Figur 2: Parametres governing the rate of deterioration

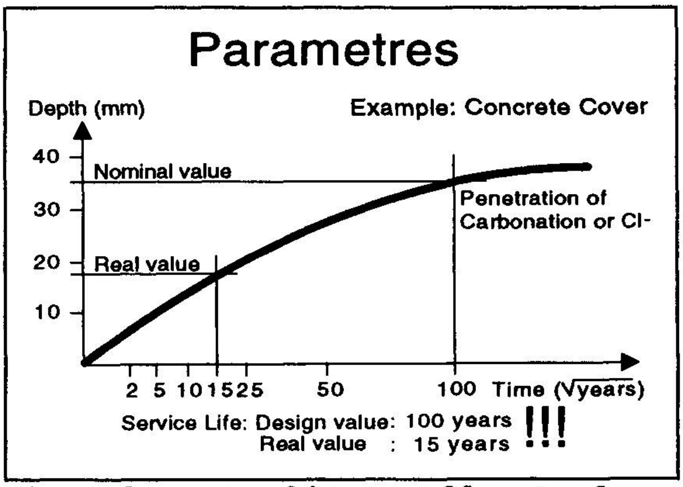
Transport Phenomena

When understanding the mechanisms in both the initiation phase and the propagation phase one very decisive fact comes clear, as illustrated on Fig.2:

All important deterioration mechanisms depend on some substance penetrating from the outside into the bulk of the concrete through the surface.

This observation is important as it highlights which zones are critical for the future performance, when designing and executing structures. Cyclic wetting and drying effects will strongly accelerate the rate at which dissolved aggressive substance enters the concrete and concentrates near the surface of evaporation.

All these transport mechanisms are non-linear by nature. This must be considered when evaluating the consequences of a given aggressive environment acting on a structure. For example, the penetration rate of a carbonation front into concrete is nearly proportional to the square-root of the exposure time. Chloride and sulphate diffusion will have a similar non-linear rate of penetration. This fact has serious practical implications, as smaller covers than anticipated in the design may lead to severely shortening of the service life, as exemplified on Fig.3.



Figur 3: Non-linear effects of governing parametres

Design

It is the prime concern during the design process to consider the transport phenomena mentioned above and ensure that they are kept under control. This is the key to designing for long service life.

Consequently, much effort shall be made to ensure an appropriate quality of the concrete in the outer layer of the concrete structures, i.e. a well compacted strong concrete "skin" is needed with low permeability, low diffusivity and without map cracking. Besides, an adequate thickness of concrete cover to the reinforcement shall be provided. These requirements emphasize the need for careful and controlled moisture curing of the structure, as well as avoiding thermo cracking by controlling the temperature profile caused by heat of hydration.

Service life depends equally on the behaviour of structural and non-structural elements. Both shall be considered during design, construction and use of the structure.

Non-structural elements such as drainage, joints, bearings, installations etc. may require specialist attention other than that of structural engineering. Particular structural components such as anchorages, couplers and deviators for prestressing tendons and their location in the structure may require special attention.

Such equipment in structures usually have a shorter service life than the structure itself, and adequate provisions for inspection, maintenance and replacement of such elements should be provided in the design.

Structures should Grow Old Gracefully

The design should consider detailing which increases self-protection and robustness of the structure against aggressive environment. This includes provisions to ensure satisfactory weathering and ageing of exposed surfaces thus allowing buildings to grow old gracefully without expensive maintenance. An appropriate selection of structural form should be ensured at an early, conceptual stage of the project.

This is a problem very much overlooked by engineers as it "does not influence safety and serviceability" in technical sence. However, it has a great influence on the public opinion and on the user of structures. The confidence in structures is much influenced by the visual appearance. The reputation of our building material suffers much from dirty and shabby looking structures, see e.g. Fig.4, and in this sence engineers should cooperate intensively with the architects who should be much more concerned with this aspect than they have been in the past. In this respect valuable works have been done in Belgium and in England.

Figur 4: Miscoloured facade due to dirt and soot

Environmental Actions

Actions on structures influencing durability and performance are chemical and physical elements of the environment which result in effects that are not considered as loads in structural design.

Environmental conditions specified in EC 2 and in ENV 206 are presented in **Table 1**.

Table 1: Exposure classes related to environmental conditions

Exposure class		Examples of environmental conditions
1 dry environment		interior of dwellings or offices[1]
2 humid environment	a without frost	- interior of buildings where humidity is high (e.g. laundries) - exterior components - components in non-aggressive soil and/or water
	b with frost	- exterior components exposed to frost - components in non-aggressive soil and/or water and exposed to frost - interior components where the humidity is high and exposed to frost
3 humid environment with frost and de-icing agents		- interior and exterior components exposed to frost and de-icing agenst
4 seawater environment	a without frost	- components completely or partially submerged in seawater, or in the splash zone - components in saturated salt air (coastal area)
	b with frost	- components partially submerged in seawater or in the splash zone and exposed to frost - components in saturated salt air and exposed to frost
The following classes may occur alone or in combination with the above classes:		
5 aggressive chemical environment[2]	a	- slightly aggressive chemical environment (gas, liquid or solid) - aggressive industrial atmosphere
	b	moderately aggressive chemical environment (gas, liquid or solid)
	c	highly aggressive chemical environment (gas, liquid or solid)

1) This exposure class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a prolonged period of time

2) Chemically aggressive environments are classified in ISO 9690. The following equivalent exposure conditions may be used:

Exposure class 5 a: ISO classification A1G, A1L, A1S
Exposure class 5 b: ISO classification A2G, A2L, A2S
Exposure class 5 c: ISO classification A3G, A3L, A3S

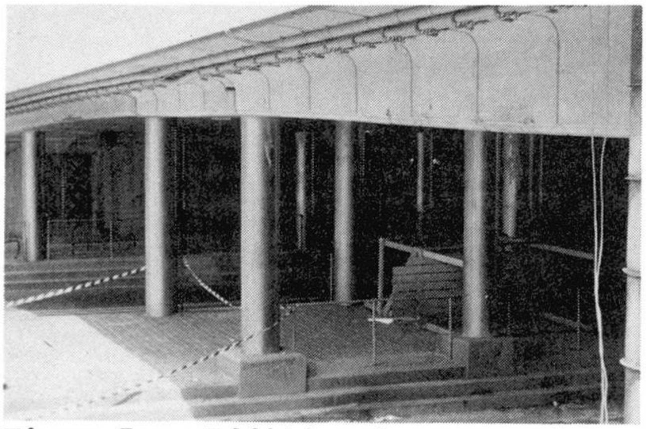
This classification covers most of the ordinary structures to be designed according to EC 2. However, some additional remarks can be valuable.

Class 1 should cover the large majority of interior concrete used in practice, but footnote 1 may cause uncertainty in some areas. This class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a prolonged period of time. In normal construction the exterior exposure will be more severe than the permanent interior condition, in some cases, e.g. with winter construction, this difference will be very pronounced. Thus the open question is "how long is a prolonged period of time"? It seems reasonable that only if there are risks of critical amounts of aggressive substance, like chlorides, to accumulate in the concrete during this exposure, or experience shows that frost damage may occur, then a more severe classification shall be used.

Class 2 covers a large number of interior and particularly exterior structures or structural components with the main limitation that there is no exposure to salt.

Class 3 covers salt exposed land based structures, whereby the majority of bridges, bridge columns, and balkonies exposed to de-icing salts, belong to this exposure class. This class may require some carefull consideration of the micro-environment, because very local concentrations of moisture, cyclic wetting and drying with salty water, salt laiden fog etc., may locally create very aggressive conditions that would require additional protection. The general or overall classification does not suffice in such cases, and the future performance will depend strongly on the competence and experience of the design engineer.

Class 4 covers all marine structures. In this respect the difference between class 3 and class 4b is small. Again the cyclic wetting and drying condition represents the most severe situation. Experience has shown that normal design provisions usually cannot ensure very long service life in these zones. Due to the especially aggressive effects of chlorides with respect to corrosion of reinforcement, and the difficulty to ensure long term reliable repairs due to the catalytic effect of chlorides, such zones should be identified in the design and be considered specially. In such situations additional protection may become necessary, see e.g. Fig.5.



Figur 5: Additional protection. Stainless steel lined reinforced concrete columns frequently exposed to de-icing salt

Class 5 covers chemical attacks from gaseous, liquid or solid substance defined by special ISO classifications. It is important to realize, that this class only covers chemical aggressivity towards concrete, and not substance that only is aggressive when reaching the reinforcement or other steel items embedded in or partially cast into concrete, but is non-aggressive to concrete.

Resistance through Design and Execution

To cope with the identified environmental actions, the following approach has been adopted.

The concrete shall be able to protect the reinforcing steel against corrosion and withstand satisfactorily the environmental and working conditions to which it is exposed during the intended lifetime. Thus the following factors have to be taken into account by the designer:

a) choice of suitable constituents
b) choice of concrete composition
c) mechanical attacks
d) mixing, placing and compaction of the fresh concrete
e) curing of the concrete

The curing shall particularly ensure that the surface zone of the structure, i.e. the cover to the reinforcement, achieves the potential properties to be expected from the chosen mix. All these factors considered shall be controlled and verified by production control by the contractor, the subcontractor or the supplier, each within his specific task, as specified in more detail in ENV 206.

Concrete

The durability requirements in ENV 206 for the fresh and hardened concrete related to the environmental exposure are presented in **Table 2.**

Table 2: Durability requirements related to environmental exposure

Requirements	Exposure class according to table 1								
	1	2a	2b	3	3a	4b	5a	5b	5c
max w/c ratio for[2)] - plain concrete - reinforced concrete - prestressed concrete	 - 0.65 0.60	 0.70 0.60 0.60	0.55	0.50	0.55	0.50	0.55	0.50	0.45
min cement content[2)] in kg/m^3 for - plain concrete - reinforced concrete - prestressed concrete	 150 260 300	 200 280 300	 200 280 300	300	300	300	 200 280 300	300	300
min air content of fresh concrete in % for nominal max aggregate size of[3)] - 32 mm - 16 mm - 8 mm	 - - -	 - - -	4) 4 5 6	4) 4 5 6	 - - -	4) 4 5 6	 - - -	 - - -	 - - -
frost resistant aggregates[6)]	-	-	yes	yes	-	yes	-	-	-
impermeable concrete according to clause 7.3.1.5	-	-	yes	yes	yes	yes	yes	yes	yes

Requirements	Exposure class according to table 1	
types of cement for plain and reinforced concrete according to EN 197		sulphate resisting cement[5] for sulphate contents > 500 mg/kg in water > 3000 mg/kg in soil
	These values of w/c ratio and cement content are based on cement where there is long experience in many countries. However, at the time of drafting this pre-standard experience with some of the cements standardized in EN 197 is limited to local climatic conditions in some countries. Therefore during the life of this pre-standard, particularly for exposure classes 2b, 3, 4b the choice of the type of cement and its composition should follow the national standards or regulations valid in the place of use of the concrete. Alternatively the suitability for use of the cements may be proved by testing the concrete under the intended conditions of use. Additionally cement CEI may be used generally for prestressed concrete. Other types of cement may be used if experience with these types is available and the apælication is allowed by the national standards or regulations valid in the place of use of the concrete.	

1) In addition, the concrete shall be protected against direct contact with the aggressive media by coatings unless for particular cases such protection is considered unnecessary.
2) For minimum cement content and maximum water/cement ratio laid down in this standard only cement listed in clause 4.1 shall be taken into account. When pozzolanic or latent hydraulic additions are added to the mix, national standards or regulations, valid in the place of use of the concrete may state if and how the minimum or maximum values respectively are allowed to be modified.
3) With a spacing factor of the entrained air void system < 0.20 mm measured on the hardened concrete
4) in cases where the degree of saturation is high for prlonged periods of time.
Other values or measures may apply if the concrete is tested and documented to have adequate frost resistance according to the national standards or regulations valid in the place of use of the concrete.
5) The sulphate resistance of the cement shall be judged on the basis of national standards or regulations valid in the place of use of the concrete.
6) Assessed against the national standards or regulations valid in the place of use of the concrete.

The cements accepted according to EN 197 have been the subject of much discussion, and the individual cement compositions adopted for the different environmental classes should be carefully considered. In particular some cements contain very large contents of pozzolanic additions making them less suited to resist some external conditions, e.g. frost and de-icing salts. Such cements are only frost resistant with the appropriate amount and distribution of entrained air. In the cements covered by EN 197 all powders in the mix are considered to have the same efficiency as cementitious material. Some countries have been used to consider e.g. microsilica more efficient than flyash (factor on strength of 2 to 0.5 respectively, when compared to Ordinary Portland Cement).

Water/cement ratio has deliberately been keept low, knowing the very strong influence this parameter has on the diffusivity of concrete. In particular the maximum value of 0.65 in exposure class 1 could be considered severe, but this will undoubtedly become advantagous in practice, as the concern about interior components exposed during construction, as mentioned above, becomes less of a worry. The maximum values specified will lead to target values being about 0.03 lower. For locally very exposed zones outside the general classification, lower values should be used, e.g. max. values of 0.35 to 0.40 would be relevant for chloride attacks.

Referring to Table 20 of ENV 206, a correlation between w/c ratio and strength class has been attempted. In practical terms this can be very convenient, and works apparently well in some cases. This is the case when considering frost resistance and resistance against carbonation which correlates well with strength classes, but this is certainly not the case when considering penetration of chlorides. Special requirements to ensure low diffusion coefficients in such cases should be provided.

Minimum cement content ensures a minimum amount of binder in the concrete in spite of the fact that strength requirements in several cases could be satisfied with much lower cement contents. This is valuable for durability, but has the drawback that it weakens the motivation to ensure good control with low variations in control parameters; strength requirements are satisfied even with sloppy control. One alternative could be to specify rather high minimum strength classes for the different exposure classes, this could keep up motivation for good quality control as this has a direct beneficial effect on the economy of the producer.

The requirements in Table 2 for minimum air content, frost resistant aggregates, impermeable concretes and types of cement seem self explanatory.

The reqirements of Table 1 in ENV 206 specifies the maximum chloride content of concrete. They are:

plain concrete: 1.0%
reinforced concrete: 0.4%
prestressed concrete: 0.2%

It is interesting to note that the value for reinforced concrete is the same as the corrosion threshold value often presented in the literature, so the ENV 206 allows no future penetration of chlorides.

Concrete cover

The concrete covers specified in EC 2 are presented in **Table 3.**

Table 3: Minimum cover requirements for normal weight concrete[1]

		Exposure class, according to table 1								
		1	2a	2b	3	4a	4b	5a	5b	(3) 5c
(2) Minimum cover (mm)	Reinforcement	15	20	25	40	40	40	25	30	40
	Prestressing steel	25	30	35	50	50	50	35	40	50

Notes:

1) In order to protect the reinforcement against corrosion, these minimum values for cover should be associated with particular concrete qualities, to be determined from Table 2 above.

2) For slab elements, a reduction of 5 mm may be made for exposure classes 2-5.

3) A reduction of 5 mm may be made where concrete of strength class C40/50 and above is used for reinforced concrete in exposure classes 2a-5b, and for prestressed concrete in exposure classes 1-5b. However, the minimum cover should never be less than that for Exposure Class 1 in Table 3 above.

4) For exposure class 5c, the use of a protective barrier, to prevent direct contact with the aggressive media, should be provided, see e.g. Fig.5.

Special care shall be taken to ensure a high quality impermeable concrete in the outer layer - or "skin" - of the structure. Casting and curing conditions have a decisive influence on the permeability of this "skin"-concrete. Experience has shown that some individual judgement of cover is needed in especially aggressive environments.

The nominal values, c_{nom}, are equal to the minimum values plus tolerance according to the following rule:

$$c_{nom} = c_{min} + \text{tolerance}$$

Tolerances are in the range of 5 to 10 mm for insitu cast formset concrete, and in the range of 0 to 5 mm for precast elements, if production conctrol can guarantee these latter values and if they are verified by appropriate quality control.

Spacers shall be designed according to c_{nom}. This fact is often overlooked! When controlling concrete cover after placing and hardening of concrete the measured values may not be less than c_{min}.

The values of cover above, quoted from EC 2, refer only to corrosion protection of the reinforcement. Other reasons may warrant larger covers such as:

- ensuring bond strength
- ensuring fire protection
- use of large aggregate sizes
- prevent spalling

In aggressive environments spacer material should preferably have good adhesion to the concrete. Requirements to concrete quality in the outer layer - or "skin" - of the structure shall also be satisfied for concrete spacers.

Curing

In order to obtain the potential properties to be expected from the concrete, especially in the surface zone, thorough curing and protection for an adequate period is necessary. Such curing and protection should start as soon as possible after the compaction of the concrete.

Guidance as to curing methods and curing times are given in ENV 206, but for more complicated cases it is recommended to perform a more detailed analysis of the curing needed, as this is very different depending on, among others, the type of cement chosen. Concretes with pozzolanic additions are very sensitive to early drying out.

In practice protection against thermal cracking of the surface is performed at the same time as the moisture curing is performed. The provisions to satisfy both requirements are thus often combined. The maximum temperature difference between the center and the surface of the hardening concrete shall be less than

20 °C. However, thermo cracking due to temperature differences across construction joints often cause more severe problems than the temperature difference between the center and the surface. Based on extensive analysis and practical experience it can be recommended for the ordinary cases to ensure a maximum temperature of 15 °C across construction joints.

Special Protective Measures

In case of especially aggressive environments where the normal provisions to ensure the required service life cannot suffice, and in cases where insufficient durability have resulted in damage to an existing structure, special protective measures may be applied to obtain the required service life.

The special protective measures are of the following type:

- Provide smooth surfaces and minimize the area exposed to the environmental aggressivity

- Provide structural protection such as:
  * roof, eaves or similar to protect concrete surfaces against rain,
  * surface protection
  * increased concrete cover. Provide special skin reinforcement if $c_{nom} \geq$ 70 mm.
  * reduce environmental aggressivity by e.g. surface insulation thus controlling heat and moisture conditions in the concrete (in buildings and housing).

- Provide special protection of the reinforcement, such as:
  * place prestressed reinforcement in sheathings (metallic or plastic) with special corrosion protective grout or void filler
  * coating of reinforcement
  * cathodic protection
  * select non-corroding reinforcement (specific stainless steel).

- Provide special monitoring systems (e.g. a warning system) to follow the condition of the structure.

- Provide intensified inspection and maintenance routines to cope with early deterioration. Implement a Management System.

With respect to epoxy coated reinforcement, this technology has only recently been introduced in Europe. The usual procedure has been to coat straight bars individually. Then cut them to length and bend them to form. This required patch repairs of the coating at cut ends and at damaged bends. When reinforcing bars are coated individually, they will not be in electric contact in the structure. This will prevent a later installation of cathodic protection, should the need arise.

The first large scale European application of epoxy coated reinforcement was for the 62,000 precast tunnel segments for the lining of the Eastern Tunnel of the Great Belt Link in Denmark. The cages were fully welded together prior to cleaning and coating, where the fluidized bed dipping technique was applied to coat the finished cages. This minimized the need for repairs and allows for cathodic protection to be applied some time in the future, should the need arise.

Gross-Error Problems, a Challenge to Education, Information and Quality Assurance

Analysing numerous cases of serious premature deterioration reveals that in the majority of cases the cause of damage is not due to normally anticipated (accepted) insitu variations in material properties, concrete covers etc. but is due to gross deviations from anticipated values. Examples are:

- 5 mm covers instead of 50 mm
- large honeycombing/bad compaction
- 150 kg cement/m^3 instead of say 350 kg
- w/c ratio = 0.75 instead of 0.45
- Design faults threatening safety (e.g. insufficient reinforcement)
- Errors in specified types of cement (e.g. high sulphate resistant cement in heavily chloride contaminated environments).

Such gross-error problems cannot be solved by stricter Eurocodes or tighter European Standards, nor by more refined design procedures, nor by the use of advanced theories of reliability. Only by enforceing relevant information and education routines and by carefully planned and strictly enforced quality assurance procedures can the frequency of such gross errors be minimized,- and our profession maintain respectability.