

Eurocode 2: Design of concrete structures —

**Part 1: General rules and rules for
buildings —**

**(together with United Kingdom
National Application Document)**

Cooperating organizations

The European Committee for Standardization (CEN), under whose supervision this European Prestandard was approved, comprises the national standards organizations of the following countries:

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United Kingdom	British Standards institution

This Draft for Development, having been prepared under the direction of the Technical Sector Board for Building and Civil Engineering (B/-), was published under the authority of the Standards Board and comes into effect on 15 May 1992

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The following BSI reference relates to the work on this Draft for Development:
Committee reference B/525/2

ISBN 0 580 20898 2

Amendments issued since publication

Amd. No.	Date	Comments

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National foreword

This publication comprises the English language version of ENV 1992-1-1:1991 *Eurocode 2: Design of concrete structures — Part 1: General rules and rules for buildings*, as published by the European Committee for Standardization (CEN), plus the National Application Document to be used with the ENV in the design of buildings to be constructed in the United Kingdom.

ENV 1992-1-1:1991 results from a programme of work sponsored by the European Commission to make available a common set of rules for the design of building and civil engineering works.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard.

The values for certain parameters in the ENV Eurocodes may be set by CEN members so as to meet the requirements of national regulations. These parameters are designated by |_| in the ENV.

During the ENV period of validity, reference should be made to the supporting documents listed in the National Application Document (NAD).

The purpose of the NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for buildings constructed in the UK, and the NAD takes precedence over corresponding provisions in the ENV.

The Building Regulations 1991, Approved Document A 1992 (published December 1991) identifies ENV 1992-1-1:1991 as appropriate guidance, when used in conjunction with this NAD, for the design of concrete buildings.

Compliance with ENV 1992-1-1:1991 and the NAD does not in itself confer immunity from legal obligations.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted to an EN.

Comments should be sent in writing to BSI, 2 Park Street, London W1A 2BS, quoting the document reference, the relevant clause and, where possible, a proposed revision, within 2 years of the issue of this document.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to xiv, the EN title page, pages 2 to 176, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

National Application Document for use in the UK with ENV 1992-1-1:1991

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Introduction

This National Application Document (NAD) has been prepared under the direction of the Technical Sector Board for Building and Civil Engineering. It has been developed from:

- a) a textual examination of ENV 1992-1-1:1991;
- b) a parametric calibration against BS 8110, supporting standards and test data;
- c) trial calculations.

1 Scope

This NAD provides information to enable ENV 1992-1-1:1991 (hereafter referred to as EC2) to be used for the design of buildings to be constructed in the UK.

2 References

2.1 Normative references

This National Application Document incorporates, by reference, provisions from specific editions of other publications. These normative references are cited at the appropriate points in the text and the publications are listed on page xiii. Subsequent amendments to, or revisions of, any of these publications apply to this National Application Document only when incorporated in it by updating or revision.

2.2 Informative references

This National Application Document refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on page xiii, but reference should be made to the latest editions.

3 Partial factors, combination factors and other values

- a) The values for combination factors (ψ) should be those given in Table 1 of this NAD.
- b) The values for partial factors should be those given in EC2 except for those given in Table 2 of this NAD.
- c) Other values should be those given in EC2 except for those given in Table 3 of this NAD.

Table 1 — Combination factors

Variable actions		ψ_0	ψ_1	ψ_2
Imposed loads	Dwellings	0.5	0.4	0.2
	Offices and stores	0.7	0.6	0.3
	Parking	0.7	0.7	0.6
Wind loads		0.7	0.2	0
Snow loads		0.7	0.2	0
NOTE For the purposes of EC2 these three categories of variable actions should be treated as separate and independent actions.				

Table 2 — Partial factors

Reference in EC2	Definition	UK values
2.3.3.1(3)	$\gamma_{G, \text{inf}}$	1.0
	$\gamma_{G, \text{sup}}$	1.35

Table 3 — Values to be used in referenced clauses instead of boxed values

Reference in EC2	Definition	UK values
2.5.1.3(4)	Assumed imperfection v (radians) when 2nd order effects are insignificant	1/200
2.5.3.5.5(5)	Limits to ratio of support to mid-span moment where plastic analysis is used	Between 1.0 and 2.0
3.2.5.1(5)	Minimum shear strength of welds	25 % of the tensile strength of the bar
4.1.3.3(8)	Allowance for tolerance (Δh) in cover for precast elements Allowance for tolerance (Δh) in cover for in situ concrete	$\Delta h = 5$ mm $\Delta h = 5$ mm
4.1.3.3, Table 4.2 and notes	Minimum covers as a function of exposure	See Table 4 of this NAD
4.2.1.3.3(12)	Reduction factor α to take account of the effects of long-term loading on maximum compressive stress (compression zone decreasing in width)	0.85
4.2.3.4.1(2)	Ratio of long-term relaxation to 1 000 h relaxation	As Table 4.6 of BS 8110-1:1985
4.3.2.4.4(1)	Limits to $\cot \theta$ in the variable strut inclination method	For all beams (with constant or curtailed reinforcement) $0.67 < \cot \theta < 1.5$
4.3.3.1(6) eqn 4.42	Limits to $\cot \theta$ in torsion calculation	$0.67 < \cot \theta < 1.5$
4.3.4.5.2(1) eqn 4.57	v_{Rd2}	2.0 (see also 6.4 d) of this NAD)
4.3.5.7(2) eqn 4.77	Limit to depth of beams	$h < 4b$
Table 5.1	Minimum diameter of mandrels	See Table 8 of this NAD
5.2.4.1.3(1)	Limiting value of the clear spacing a above which α_1 may take a value of 1.0 for compression and 1.4 for tension Limiting value of b to lapped bar above which α_1 may take a value of 1.0 for compression and 1.4 for tension	6ϕ 2ϕ
5.2.5(3)	Extent of bar beyond bend in link	4ϕ instead of 5ϕ 8ϕ instead of 10ϕ
5.4.1.2.2(4)	Factor by which minimum spacing should be reduced under defined circumstances In item ii), bar size near lap above which spacing of transverse steel should be reduced	0.67 20 mm
5.4.3.2.1(4)	Maximum bar spacing in slab	Principal and secondary reinforcement $3h \nlessgtr 500$ mm
5.4.3.3(2)	Minimum shear reinforcement in slabs as a percentage of the value for beams	100 %
5.4.8.1(3)	Factor defining maximum local stress under a concentrated load	See note below
6.2.2(1)	Tolerances	See 6.6 of this NAD
NOTE Equation 5.22 should read: $F_{Rdu} = A_{co} a f_{cd} \sqrt{(A_{e1}/A_{co})} \nlessgtr 3.3 a f_{cd} A_{co}$ where α is as defined in 4.2.1.3.3 (11) of EC2		

4 Loading codes

The loading codes to be used are:

BS 648:1964, *Schedule of weights of building materials*.

BS 6399, *Loading for buildings*.

BS 6399-1:1984, *Code of practice for dead and imposed loads*.

BS 6399-3:1988, *Code of practice for imposed roof loads*.

CP 3, *Code of basic data for the design of buildings*.

CP 3:Chapter V, *Loading*.

CP 3:Chapter V-2:1972, *Wind loads*.

In using the above documents with EC2 the following modifications should be noted.

- a) The imposed floor loads of a building should be treated as one variable action to which the reduction factors given in BS 6399-1:1984 are applicable.
- b) Snow drift loads obtained from BS 6399-3:1988 should not be treated as accidental actions as defined in EC2. They should be multiplied by 0.7 and treated as a variable action.
- c) The wind loading should be taken as 90 % of the value obtained from CP 3:Chapter V-2:1972.

5 Reference standards

Standards including materials specifications and standards for construction are listed for reference in Table 4 of this NAD.

Table 4 — References in EC2 to other publications

Reference in EC2	Document referred to	Document title or subject area	Status	UK document
1.1.1P(4)	Eurocode 8	Design of structures in seismic regions	Draft	—
1.1.1P(5)	Eurocode 1	Actions on structures	Partially drafted	BS 6399-1 and BS 6399-3 CP3:Chapter V-2
1.1.3P(2)	EC2-1A	Plain or lightly reinforced concrete structures	1st draft	3.8.1.4 and 3.9.4 of BS 8110-1:1985
1.1.3P(2)	EC2-1B	Precast concrete structures	1st draft	Section 5 of BS 8110-1:1985
1.1.3P(2)	EC2-1C	The use of lightweight aggregate concrete	1st draft	Section 5 of BS 8110-2:1985
1.1.3P(2)	EC2-1D	The use of unbonded and external prestressing tendons	1st draft	Section 4 of BS 8110-1:1985 (See note 1)
1.1.3P(2)	EC2-1E	Fatigue requirements	No draft	BS 8110 BS 5400-10 (See note 3)
1.1.3P(2)	EC2-2	Reinforced and prestressed concrete bridges	Work in progress	BS 5400-4
1.1.3P(2)	EC2-3	Concrete foundations and piling	Work not started	BS 8004
1.1.3P(2)	EC2-4	Liquid-retaining structures now renamed Retaining and containment structures	Work not started	BS 8007
1.1.3P(2)	EC2-5	Temporary structures, structures having a short design life	Work not started	—
1.1.3P(2)	EC2-6	Massive civil engineering structures	Work not started	—
1.1.3P(2)	EC2-10	Fire resistance of concrete structures	Work in progress	3.3.6 of BS 8110-1:1985 Section 4 of BS 8110-2:1985
1.4.1	ISO 8930	General principles on reliability of structures	Published 1987	BS 6100
	ISO 6707-1	Building and civil engineering — General vocabulary — Part 1	Published 1989	BS 6100
1.5	ISO 1000	SI Units and recommendations for the use of their multiples and of certain other units	Published 1981	BS 5555

Table 4 — References in EC2 to other publications

Reference in EC2	Document referred to	Document title or subject area	Status	UK document
3.1.1P(1)	ENV 206 and Annex	Concrete-performance, production, placing and compliance criteria	Published 1989	BS 5328-1, BS 5328-2, BS 5328-3 and BS 5328-4. 3.3.5 and section 6 of BS 8110-1:1985 (See note 3)
3.1.1(4)	5.2 of ENV 206	Air content limits	Published 1989	6.2.3.2 of BS 8110-1:1985
3.1.2.1	7.3.2 of ENV 206	Density classes of concrete	Published 1989	6.3.5.3 of BS 8110-1:1985
3.1.2.2(2)	7.3.1.1 of ENV 206	Tests for compressive strength	Published 1989	BS 1881-108, BS 1881-110, BS 1881-111 and BS 1881-116 BS 5328-1 and BS 5328-2
3.1.2.3P(2)	7.3.1.2 of ENV 206	Tensile strength	Published 1989	BS 1881-118 BS 5328-1 and BS 5328-2
3.1.2.4(2)	7.3.1.1 and 11.3.5.4 of ENV 206	Strength classes	Published 1989	BS 5328-1
3.1.2.5.5(4)	7.2.1 of ENV 206	Consistency of wet concrete	Published 1989	BS 1881-102 BS 5328-1 and BS 5328-4
3.2.1(3)	prEN 10080	Standard for reinforcement	prEN subject to CEN enquiry	BS 4449, BS 4482 and BS 4483
3.2.1P(5)	Relevant standards	Strength and geometrical properties of bar tolerances on cross-sectioned area. Dimensional requirements for welded mesh ratio f_y/f_{yk}	—	BS 4449, BS 4482 and BS 4483
3.2.4.2P(1)	Relevant standards	Ductility and elongation requirements	—	Clause 10 of BS 4449:1988 and 12.1.3 of BS 4482:1985
3.2.4.4P(1)	Relevant standards	Fatigue requirements for reinforcement	—	Clause 11 and Appendix D of BS 4449:1988
3.3 and throughout	prEN 10138-1 to prEN 10138-5 Relevant standards	Prestressing steel	Out for public comment	BS 5896, BS 4486 and BS 4756
3.4	Relevant standards European Approval Documents	Anchorage	—	BS 4447
4.1.2.1	Appropriate international or national codes	Actions on structures EC1	Being drafted	See clause 4 of this NAD
4.1.3.3 and Table 4.1	ISO/DP 9690	Classification of environmental conditions for concrete structures	—	—
4.2.3.4.1	Relevant standards	Relaxation of prestressing steel	—	BS 5896, BS 4486 and BS 4756
4.4.3.1(4)	ISO 4356	Basis for the design of structures — Deformation of buildings at the serviceability limit states	Published Accepted by UK	3.2.1 and 3.2.2 of BS 8110-2:1985
6.3.2.2	Appropriate national or international documents	Specification of finishes	—	6.10 of BS 8110-1:1985

Table 4 — References in EC2 to other publications

Reference in EC2	Document referred to	Document title or subject area	Status	UK document
6.3.3.1	Relevant Euronorms or CEN, ISO or national standards, National Building Regulations Control Authority	Requirements for reinforcing steel	—	BS 4449, BS 4482 and BS 4483
6.3.3.2P(3)	Appropriate international or national standards	Cutting and bending of reinforcement	—	BS 4466
6.3.3.3P(4)	Relevant standards	Fatigue requirements for welding of reinforcement	—	BS 5400-10 (See note 2)
6.3.3.3P(5)	International or national standards	Production and checking of welded connections	—	BS 7123
6.3.3.4P(5)	Standards or approval documents	Mechanical connectors	—	3.12.8.16 of BS 8110-1:1985 (See note 4)
6.3.4.6.2P(4)	See EN 447	Types of cement for grouting	—	BS 12
7.2.1	Eurocode I	Required level of quality control	Work in progress	6.4 and sections 7 and 8 of BS 8110-1:1985 BS 5328-1 and BS 5328-4
7.5	CEC or National Administrative Procedures	Control of design	—	Building Regulations
7.6.5.1P(6)	Relevant technical documents	All other structural materials	Other Eurocodes	—

NOTE 1 Unbonded tendons are not treated in any depth in BS 8110 but EC2 Section 4 applies.

NOTE 2 It is assumed that fatigue will not be a problem in buildings and hence BS 8110 does not give any rules.

NOTE 3 The exposure conditions referred to in 4.1.3.3 are related to exposure to chemically aggressive environments (e.g. industrial pollutants) not normal environments. There is no UK equivalent document.

NOTE 4 Clause 3.12.8.16 of BS 8110-1:1985 on butt joints gives rules for the use of both tension and compression couplers. BS 8110-1:1985 Clause 3.12.8.16.2 gives performance criteria for tension couplers.

6 Additional recommendations

6.1 Chapter 1. Scope

a) Clause 1.1.2, item P(5)

Resistance to fire should be in accordance with BS 8110-2:1985.

6.2 Chapter 2. Basis of design

a) Clause 2.3.2.3(4)

Item (4) applies to all permanent actions.

b) Clause 2.3.3, Table 2.2

It should be assumed that the favourable effects are $\gamma_{F, inf}$ and that the unfavourable effects are $\gamma_{F, sup}$. Footnote *) and 2.3.3.1(3) need not be taken into account since the boxed values given in Table 2 of this NAD are the same as those given in Table 2.2 of EC2.

In footnote ***) the relevant clause is 2.5.4.4.3(3).

c) Clause 2.3.3.1

In addition to item b), the partial safety factor γ_Q for earth and water pressures should be 1.35 and the recommendations given in clause 2.4.3.1.2 of BS 8110-1:1985 should be followed.

d) Clause 2.3.3.1(8)

In the expressions 2.8(a) and 2.8(b), the boxed figures represent γ_Q values.

When using the expression 2.8(b) for independent variable actions, the loading represented by the expression 2.8(a) should also be considered for each independent action.

e) *Clause 2.5.1.2*

Normally it will be sufficient to consider the combination of dead, imposed and wind loads, without patterning the imposed loads, i.e. expression 2.8 b) in 2.3.3.1(8) may be used. However, load patterning may need to be considered in sensitive structures.

6.3 Chapter 3. Material propertiesa) *Clause 3.2*

Current British Standards for reinforcement are BS 4449:1988 (bars) and BS 4483:1985 (welded fabric). It is envisaged that when EN 10080 (currently in draft form as prEN 10080) is published Grade 500 (bars and fabric) will supersede Grade 460.

The differences between the current British Standards and prEN 10080 are summarized in Table 5.

6.4 Chapter 4. Section and member designa) *Clause 4.1.3.3(5)*

Item (5) should be replaced by the following:

“(5) The specified nominal cover to all reinforcement should be not less than the size of the main bar or, where bars are in pairs or bundles, the size of a single bar of the cross-sectional area equal to the sum of their cross-sectional areas. It should be not less than the nominal maximum size of the aggregate.”

b) *Clause 4.1.3.3(6)*

The specified nominal cover required for reinforcement, including links, to ensure adequate durability should be in accordance with Table 6 of this NAD. To conform with current UK practice, Table 6 gives nominal values of cover, which include allowance for the tolerance of 5 mm.

c) *Table 4.8*

The maximum value of f_{ck} in the determination of τ_{Rd} should be taken as 40 N/mm² (strength class C40/50).

d) *Clause 4.3.4.5.2(1)*

In equation 4.57, in addition to the limitation on v_{Rd2} given in Table 3 of this NAD the shear stress at the perimeter of the column should not exceed $0.9 \sqrt{f_{ck}}$

Equation 4.58 is applicable where $v_{Rd3} \leq 1.6v_{Rd1}$. Where $1.6v_{Rd1} < v_{Rd3} \leq 2.0v_{Rd1}$ equation 4.58a below should be used:

$$v_{Rd3} = 1.4v_{Rd1} + (0.3 \Sigma A_{sw} f_{ya} \sin \alpha) / u \quad (4.58a)$$

e) *Clause 4.4.3.2(5)*

At the end of c) the following sentence should be added:

““Nominally reinforced” values to correspond to $\rho = 0.15\%$.”

f) *Table 4.14*

Table 4.14 should be replaced by Table 7 of this NAD.

Table 5 — Differences between current British Standards and prEN 10080

Property	BS 4449 and BS 4483	prEN 10080
Specified characteristic yield strength	Grade 460 N/mm ² Grade 250 N/mm ²	500 N/mm ² not included
Bond strength for:		
Ribbed bars/wires	Deformed Type 2	High bond
Indented wires	Deformed Type 1	Not included
Plain bars/wires	Plain rounds	Not included
Ductility class (now defined as elongation at maximum load and ultimate to yield strength ratio)	Not covered	Class H or Class N (this may be deleted in the final version) (see note)
NOTE All ribbed bars and all Grade 250 bars may be assumed to be Class H. Ribbed wire welded fabric may be assumed to be available in Class H in wire sizes of 6 mm or over. Plain or indented wire welded fabric may be assumed to be available in Class N. In design where plastic analysis or moment distribution over 15 % is used, it is essential to specify ductility class (H) as defined in prEN 10080 since this parameter is not covered by BS 4449 and BS 4483.		

Table 6 — Cover requirements for normal weight concrete

		Exposure class, according to Table 4.1								
		1	2a	2b	3	4a	4b	5a	5b	5c
Reinforcement	Nominal ^a	20	35	35	40	40	40	35	35	45
	Minimum	(15)	(30)	(30)	(35)	(35)	(35)	(30)	(30)	(40)
Prestressing	Nominal ^a	25	40	40	45	45	45	40	40	50
	Minimum	(20)	(35)	(35)	(40)	(40)	(40)	(35)	(35)	(45)
NOTE 1 In order to satisfy the provisions of 4.1.3.3P(3) these values for cover should be associated with particular concrete qualities, to be determined from Table 3 of ENV 206 and its National Annex.										
NOTE 2 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 to 5b, and for prestressed concrete in exposure classes 2a to 5b. For slab elements, a further reduction of 5 mm may be made for exposure classes 2 to 5.										
NOTE 3 For exposure class 5c, a protective barrier should be provided to prevent direct contact with aggressive media.										
^a The nominal values for cover have been obtained from the minimum values by allowing for a negative construction tolerance of 5 mm.										

Table 7 — Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural system	Concrete highly stressed	Concrete lightly stressed	Concrete nominally reinforced
1. Simply supported beam, one or two-way spanning simply supported slab	18	25	34
2. End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	23	32	44
3. Interior span of beam or one-way or two-way spanning slab	25	35	38
4. Slab supported on columns without beams (Flat slab), based on longer span	21	30	41
5. Cantilever	7	10	14
NOTE 1 See also 6.4 e) of this NAD.			
NOTE 2 4.4.3.2(4) of EC2 should not be applied to the basic span/effective depth ratios for nominally reinforced concrete.			

6.5 Chapter 5. Detailing provisions

a) Table 5.1

Table 5.1 should be replaced by Table 8 of this NAD which gives minimum diameters of mandrels.

b) Clause 5.2.4.1.2

This clause should be interpreted as applying to laps in reinforcement in beams only.

c) Clause 5.4.2.1.4(3)

When using equation 5.4 within this clause [see 5.2.3.4.1(1)] $A_{s,req}$ should be at least one-quarter of the area of the tension reinforcement in the span [see 5.4.2.1.4(1)].

d) Clause 5.4.2.4(2)

This reinforcement should be placed inside the stirrups.

e) Clause 5.4.3.2.3

When corner reinforcement is required, the conditions in 3.5.3.5, 3.5.3.6 and 3.5.3.7 of BS 8110-1:1985 should be used.

f) Clause 5.4.3.3(4)

Longitudinal spacing of successive links should not exceed $0.75d$.

g) Clause 5.5.2

The provision of ties should be based on 2.4.3.2, 3.12.3 and 5.1.8 of BS 8110-1:1985 and 2.6 of BS 8110-2:1985.

6.6 Chapter 6. Construction and workmanship

a) Clause 6.2

Tolerances in this clause should be read as dimensional deviations and should be based on those given in 6.11, 7.3 and 8.6 of BS 8110-1:1985.

b) Clause 6.3.3.3

Additional guidance is given in 3.12.8.16.1, 3.12.8.16.2, 3.12.8.17, 3.12.8.18 and 7.6 of BS 8110-1:1985.

Table 8 — Minimum diameters of mandrels

	Hooks, bends, loops (see Figure 5.2 of EC2)		Bent-up bars or other curved bars		
	Bar diameter		Value of minimum concrete cover, perpendicular to plane of curvature		
	$\phi < 20 \text{ mm}$	$\phi \geq 20 \text{ mm}$	$> 100 \text{ mm}$ and $> 7\phi$	$> 50 \text{ mm}$ and $> 3\phi$	$\leq 50 \text{ mm}$ and $\leq 3\phi$
Minimum diameter of mandrels for plain bars S250	4ϕ	4ϕ	7ϕ	8.5ϕ	11.4ϕ
Minimum diameter of mandrels for high bond bars S460	6ϕ	8ϕ	13ϕ	15.7ϕ	20.9ϕ

List of references (see clause 2)

Normative references

BSI standards publications

BRITISH STANDARDS INSTITUTION, London

- BS 648:1964, *Schedule of weights of building materials*.
 BS 6399, *Loading for buildings*.
 BS 6399-1:1984, *Code of practice for dead and imposed loads*.
 BS 6399-3:1988, *Code of practice for imposed roof loads*.
 BS 8110, *Structural use of concrete*.
 BS 8110-1:1985, *Code of practice for design and construction*.
 BS 8110-2:1985, *Code of practice for special circumstances*.
 CP3, *Code of basic data for the design of buildings*.
 CP3:Chapter V, *Loading*.
 CP3:Chapter V-2, *Wind loads*.

Informative references

BSI standards publications

BRITISH STANDARDS INSTITUTION, London

- BS 12:1991, *Specification for Portland cement*.
 BS 1881, *Testing concrete*.
 BS 1881-102:1983, *Method for determination of slump*.
 BS 1881-108:1983, *Method for making test cubes from fresh concrete*.
 BS 1881-110:1983, *Method for making test cylinders from fresh concrete*.
 BS 1881-111:1983, *Method of normal curing of test specimens (room temperature)*.
 BS 1881-116:1983, *Method for determination of compressive strength of concrete cubes*.
 BS 1881-118:1983, *Method for determination of flexural strength*.
 BS 4447:1973, *Specification for the performance of prestressing anchorages for post-tensioned construction*.
 BS 4449:1988, *Specification for carbon steel bars for the reinforcement of concrete*.
 BS 4466:1989, *Specification for scheduling, dimensioning, bending and cutting of steel reinforcement for concrete*.
 BS 4482:1985, *Specification for cold reduced steel wire for the reinforcement of concrete*.
 BS 4483:1985, *Specification for steel fabric for the reinforcement of concrete*.
 BS 4486:1990, *Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete*.
 BS 4756:1971, *Specification for ready mixed aluminium priming paints for woodwork*.
 BS 5328, *Concrete*.
 BS 5328-1:1991, *Guide to specifying concrete*.
 BS 5328-2:1991, *Methods for specifying concrete mixes*.
 BS 5328-3:1990, *Specification for the procedures to be used in producing and transporting concrete*.
 BS 5328-4:1990, *Specification for the procedures to be used in sampling, testing and assessing compliance of concrete*.
 BS 5400, *Steel, concrete and composite bridges*.
 BS 5555:1978, *Specification for SI units and recommendations for the use of their multiples and of certain other units*.
 BS 5896:1980, *Specification for high tensile steel wire and strand for the prestressing of concrete*.

BS 6100, *Glossary of building and civil engineering terms.*

BS 7123:1989, *Specification for metal arc welding of steel for concrete reinforcement.*

BS 8004:1986, *Code of practice for foundations.*

BS 8007:1987, *Code of practice for design of concrete structures for retaining aqueous liquids.*

CEN and CENELEC standards publications

EUROPEAN COMMITTEE FOR STANDARDIZATION (CEN) and EUROPEAN COMMITTEE FOR ELECTROTECHNICAL STANDARDIZATION (CENELEC), Brussels. (All publications are available from BSI Sales.)

ENV 206, *Concrete — Performance, production, placing and compliance criteria.*

prEN 10080, *Steel for the reinforcement of concrete weldable ribbed reinforcing steel B 500 — Technical delivery conditions for bars, coils and welded fabric.*

UDC 624.92.012.3/.4:624.07

Descriptors: Buildings, concrete structure, computation, building codes, rules of calculation

English version

Eurocode 2: Design of concrete structures — Part 1: General rules and rules for buildings

Eurocode 2: Calcul des structures en béton —
Partie 1: Règles générales et règles pour les
bâtiments

Eurocode 2: Planung von Stahlbeton- und
Spannbetontragwerken — Teil 1: Grundlagen
und Anwendungsregeln für den Hochbau

This European Prestandard (ENV) was approved by CEN on 1991-12-27 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into a European Standard (EN).

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

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CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

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Foreword

0.1 Objectives of the Eurocodes

- 1) The Structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.
- 2) They are intended to serve as reference documents for the following purposes:

- a) As a means to prove compliance of building and civil engineering works with the essential requirements of the Construction Products Directive (CPD)
- b) As a framework for drawing up harmonised technical specifications for construction products.

3) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.

4) Until the necessary set of harmonised technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

0.2 Background to the Eurocode programme

1) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonised technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

2) In 1990, after consulting their respective Member States, the CEC transferred work of further development, issue and updates of the Structural Eurocodes to CEN and the EFTA Secretariat agreed to support the CEN work.

3) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

0.3 Eurocode programme

1) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991	Eurocode 1	Basis of design and actions on structures
EN 1992	Eurocode 2	Design of concrete structures
EN 1993	Eurocode 3	Design of steel structures

EN 1994 Eurocode 4 Design of composite steel and concrete structures

EN 1995 Eurocode 5 Design of timber structures

EN 1996 Eurocode 6 Design of masonry structures

EN 1997 Eurocode 7 Geotechnical design

EN 1998 Eurocode 8 Design provisions for earthquake resistance of structures

In addition the following may be added to the programme:

EN 1999 Eurocode 9 Design of aluminium structures

2) Separate sub-committees have been formed by CEN/TC 250 for the various Eurocodes listed above.

3) This part of the Structural Eurocode for Design of Concrete Structures, which had been finalised and approved for publication under the direction of CEC, is being issued by CEN as a European Prestandard (ENV) with an initial life of three years.

4) This Prestandard is intended for experimental practical application in the design of the building and civil engineering works covered by the scope as given in 1.1.2 and for the submission of comments.

5) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future action.

6) Meanwhile, feedback and comments on this Prestandard should be sent to the Secretariat of sub-committee CEN/TC 250/SC 2 at the following address:

DIN

Burggrafenstrasse 6

D-1000 Berlin 30

GERMANY

or to your national standards organization.

0.4 National Application Documents

1) In view of the responsibilities of authorities in member countries for the safety, health and other matters covered by the essential requirements of the CPD, certain safety elements in this ENV have been assigned indicative values which are identified by | ____ |. The authorities in each member country are expected to assign definitive values to these safety elements.

2) Many of the harmonised supporting standards, including the Eurocodes giving values for actions to be taken into account and measures required for fire protection, will not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving definitive values for safety elements, referencing compatible supporting standards and providing national guidance on the application of this Prestandard, will be issued by each member country or its Standards Organisation.

3) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works are located.

0.5 Matters specific to this Prestandard

1) The scope of Eurocode 2 is defined in 1.1.1 and the scope of this Part of Eurocode 2 is defined in 1.1.2. Additional Parts of Eurocode 2 which are planned are indicated in 1.1.3; these will cover additional technologies or applications, and will complement and supplement this Part.

2) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions and conditions given in 1.3.

3) The seven chapters of this Prestandard are complemented by four Appendices which have the same normative status as the chapters to which they relate. These Appendices have been introduced by moving some of the more detailed principles/Application Rules, which are needed in particular cases, out of the main part of the text to aid its clarity.

4) As indicated in 0.4 2) of this Foreword, reference should be made to National Application Documents which will give details of compatible supporting standards to be used. For this Part of Eurocode 2, particular attention is drawn to the approved Prestandard ENV 206 (Concrete — performance, production, placing and compliance criteria), and the durability requirements given in 4.1 of this Prestandard.

5) The provisions of this Prestandard are based substantially on the 1978 edition of the CEB Model Code and other more recent CEB and FIP documents.

6) In developing this Prestandard, background documents have been prepared, which give commentaries on and justifications for some of the provisions in this Prestandard.

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1 Introduction

1.1 Scope

1.1.1 Scope of Eurocode 2

P(1) Eurocode 2 applies to the design of buildings and civil engineering works in plain, reinforced and prestressed concrete. It is subdivided into various separate parts, see 1.1.2 and 1.1.3.

P(2) This Eurocode is only concerned with the requirements for resistance, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound, insulation, are not considered.

P(3) Execution¹⁾ 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. Execution and workmanship are covered in Chapters 6 and 7, and are to be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works¹⁾ and methods of construction¹⁾.

P(4) Eurocode 2 does not cover the special requirements of seismic design. Provisions related to such requirements are given in Eurocode 8 “*Design of Structures in Seismic Regions*”²⁾ which complements, and is consistent with, Eurocode 2.

P(5) Numerical values of the actions on buildings and civil engineering works to be taken into account in the design are not given in Eurocode 2. They are provided in the Eurocode 1 “*Bases of Design and Actions on Structures*”²⁾ applicable to the various types of construction.

1.1.2 Scope of Part 1 of Eurocode 2

P(1) Part 1 of Eurocode 2 gives a general basis for the design of buildings and civil engineering works in reinforced and prestressed concrete made with normal weight aggregates (see 1.1.3 for supplementary Parts covering additional methods of construction, materials, and type of structure).

P(2) 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; their use and any limits of applicability are explained in the text where necessary.

P(3) The following subjects are dealt with in Part 1.

- Chapter 1: Introduction;
- Chapter 2: Basis of Design;
- Chapter 3: Material Properties;
- Chapter 4: Section and Member Design;
- Chapter 5: Detailing Provisions;
- Chapter 6: Construction and Workmanship.
- Chapter 7: Quality Control.
- Appendix 1: Time-dependent Effects.
- Appendix 2: Non-linear Analysis.
- Appendix 3: Additional Design Procedures for Buckling.
- Appendix 4: Checking deflections by calculation.

P(4) Chapters 1 and 2 are common to all Eurocodes, with the exception of some additional clauses which are required for concrete.

P(5) This Part 1 does not cover:

- resistance to fire;
- particular aspects of special types of building (such as tall buildings);
- particular aspects of special types of civil engineering works (such as viaducts, bridges, dams, pressure vessels, offshore platforms or liquid-retaining structures);
- no-fines concrete and aerated concrete components, and those made with heavy aggregate or containing structural steel sections (see Eurocode 4 for composite steel-concrete structures).

¹⁾ For the meaning of this term, see 1.4.1(2)

²⁾ At present at the draft stage.

1.1.3 Further Parts of Eurocode 2

P(1) This Part 1 of Eurocode 2 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 certain other aspects of design which are of general practical importance.

P(2) Further Parts of Eurocode 2 which, at present, are being prepared or are planned, include the following (listed in 2 distinct categories):

- *Part 1A: Plain or lightly reinforced concrete structures;*
- *Part 1B: Precast concrete structures;*
- *Part 1C: The use of lightweight aggregate concrete;*
- *Part 1D: The use of unbonded and external prestressing tendons;*
- *Part 1E: Design for fatigue of concrete structures;*
-
- *Part 10: Fire resistance of concrete structures;*
- *Part 2: Reinforced and Prestressed Concrete Bridges;*
- *Part 3: Concrete Foundations and Piling;*
- *Part 4: Liquid-retaining Structures;*
- *Part 5: Temporary Structures. Structures having a Short Design Life;*
- *Part 6: Massive Civil Engineering Structures.*

1.2 Distinction between Principles and Application Rules

P(1) Depending on the character of the individual clauses, distinction is made in this eurocode between Principles and Application Rules.

P(2) The Principles comprise:

- general statements and definitions for which there is no alternative, as well as
- requirements and analytical models for which no alternative is permitted unless specifically stated.

P(3) In this code the Principles are preceded by the letter P.

P(4) The Application Rules are generally recognized rules which follow the Principles and satisfy their requirements.

P(5) It is permissible to use alternative design rules different from the Application Rules given in the Eurocode, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the resistance, serviceability and durability achieved for the structure with the present Eurocode.

P(6) In this code Application Rules are off-set to the right.

1.3 Assumptions

P(1) The following assumptions apply:

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided in factories, in plants, and on site.
- Construction is carried out by personnel having the appropriate skill and experience.
- The construction materials and products are used as specified in this Eurocode or in the relevant material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.

P(2) The design procedures are valid only when the requirements for execution and workmanship given in Chapters 6 and 7 are also complied with.

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

1.4 Definitions

1.4.1 Terms common to all eurocodes

P(1) Unless otherwise stated in the following, the terminology used in International Standard ISO 8930 applies.

P(2) The following terms are used in common for all Eurocodes with the following meanings:

— **Construction works:** Everything that is constructed or results from construction operations.³⁾ This term covers both building and civil engineering works. It refers to the complete construction comprising both structural and non-structural elements.

— **Execution:** The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

NOTE In English “construction” may be used in certain combinations of words, when there is no ambiguity (e.g. “during construction”).

— **Structure:** Organised combination of connected parts designed to provide some measure of rigidity.⁴⁾ This term refers to load carrying parts.

— **Type of building or civil engineering works:** Type of “construction works” designating its intended purpose, e.g. dwelling house, industrial building, road bridge.

NOTE “Type of construction works” is not used in English.

— **Form of structure:** Structural type designating the arrangement of structural elements, e.g. beam, triangulated structure, arch, suspension bridge.

— **Construction material:** A material used in construction work, e.g. concrete, steel, timber, masonry.

— **Type of construction:** Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction.

— **Method of construction:** Manner in which the construction will be carried out, e.g. cast in place, prefabricated, cantilevered.

— **Structural system:** The load bearing elements of a building or civil engineering works and the way in which these elements are assumed to function, for the purpose of modelling.

P(3) The equivalent terms in the Community languages are given in Table 1.1.

1.4.2 Special terms used in Part 1 of Eurocode 2

P(1) **Plain or lightly reinforced concrete.** Reinforced concrete elements, having percentages of longitudinal flexural reinforcement less than the minimum values given in 5.4, should be treated as plain concrete and designed in accordance with Part 1A of Eurocode 2.

P(2) **Unbonded and external tendons.** For post-tensioned members, where the ducts are permanently ungrouted, and for tendons external to the concrete cross-section (which may be encased in concrete after stressing, or have a protective membrane), the design should accord with the provisions of Part 1D of Eurocode 2. Where tendons are temporarily ungrouted during construction, the provisions of Part 1 apply.

³⁾ This definition accords to the International Standard ISO 6707-1.

⁴⁾ The International Standard ISO 6707-1 gives the same definition, however, adds “or a construction works having such an arrangement”. For Eurocodes this addition is not used, in order to avoid ambiguous translations.

Table 1.1 — List of equivalent terms in Community languages (to be completed for other Community languages)

ENGLISH	FRANCAIS	DEUTSCH	ITALIANO	NEDERLANDS	ESPAÑOL
Construction works	Construction	Bauwerk	Costruzione	Bouwwerk	Construcción
Execution	Exécution	(Bau-) Ausführung	Esecuzione	Uitvoering	Ejecucion
Structure	Structure	Tragwerk	Struttura	Draag-constructie	Estructura
Type of building or civil engineering works	Nature de construction	Art des Bauwerks	Tipo di costruzione	Type bouwwerk	Naturaleza de la construcción
Form of structure	Type de structure	Art des Tragwerks	Tipo di struttura	Type draag-constructie	Tipo de estructura
Construction material	Matériau de construction	Baustoff; Werkstoff (Stahlbau)	Materiale da costruzione	Constructie materiaal	Material de construccion
Type of construction	Mode de construction	Bauart	Sistema costruttivo	Bouwwijze	Modo de construcción
Method of construction	Procédé d'exécution	Bauverfahren	Procedimento esecutivo	Bouwmethode	Procedimiento de ejecucion
Structural system	Système structural	Tragsystem	Sistema strutturale	Constructief systeem	Sistema estructural

1.5 S.I. Units

P(1) S.I. Units shall be used in accordance with ISO 1000.

(2) For calculations, the following units are recommended:

- forces and loads : kN, kN/m, kN/m²
- unit mass : kg/m³
- unit weight : kN/m³
- stresses and strengths: N/mm² (= MN/m² or MPa)
- moments (bending ...) : kNm

1.6 Symbols common to all Eurocodes

1.6.1 Latin upper case letters

A	Accidental action
A	Area
C	Fixed value
E	Modulus of elasticity
E	Effect of action
F	Action
F	Force
G	Permanent action
G	Shear modulus
I	Second moment of area
M	Moment in general
M	Bending moment
N	Axial force
P	Prestressing force
Q	Variable action
R	Resistance
S	Internal forces and moments
T	Torsional moment
V	Shear force
W	Section modulus
X	Value of a property of a material

1.6.2 Latin lower case letters

a	Distance
a	Geometrical data
	Δa Additive or reducing safety element for geometrical data
b	Width
d	Diameter; Depth
e	Eccentricity
f	Strength (of a material)

h	Height
i	Radius of gyration
k	Coefficient; Factor
l (or ℓ or L)	Length; Span ^a
m	Mass
r	Radius
t	Thickness
u,v,w	Components of the displacement of a point
x,y,z	Coordinates

^a l can be replaced by L or ℓ (handwritten) for certain lengths or to avoid confusion with 1 (numeral)

1.6.3 Greek lower case letters

α	Angle; Ratio
β	Angle; Ratio
γ	Partial safety factor
ε	Strain
λ	Slenderness ratio
μ	Coefficient of friction
ν	Poisson's ratio
ρ	Mass density
σ	Normal stress
τ	Shear stress
ψ	Factors defining representative values of variable actions
	ψ_0 for combination values
	ψ_1 for frequent values
	ψ_2 for quasi-permanent values

1.6.4 Subscripts

a	Structural steel
c	Concrete
c	Compression
cr (or crit)	Critical
d	Design
dst	Destabilising
dir	Direct
eff	Effective
ext	External
f	Flange
F (or P)	Action
g (or G)	Permanent action
h	High; Higher

ind	Indirect
inf	Inferior; Lower
int	Internal
k	Characteristic
l	Low; Lower
m (or M)	Material
m	Bending
m	Mean
max	Maximum
min	Minimum
nom	Nominal
p (or P)	Prestressing force
pl	Plastic
ps	Prestressing steel
q (or Q)	Variable action
R	Resistance
rep	Representative
s	Reinforcing steel
S	Internal moments and forces
stb	Stabilising
sup	Superior; Upper
t (or ten)	Tension
t (or tor)	Torsion
u	Ultimate
v	Shear
w	Web
x,y,z	Coordinates
y	Yield

1.7 Special symbols used in this Part 1 of Eurocode 2

1.7.1 General

In general, the symbols used in Part 1 of Eurocode 2 are based on the common symbols in 1.6 and on derivatives of these as, for example,

$G_{d,sup}$	Upper design value of a permanent action
A_c	Total cross-sectional area of a concrete section
f_{yd}	Design yield strength of reinforcement.

Such derivations are defined in the text where they occur, for ease of use. However, in addition, the most frequently occurring symbols are listed and defined below; those that are local, and unique to a particular chapter, are listed at the beginning of that chapter. If, in reading the text, the user is unsure of the meaning of a particular symbol, he will be able to find it either in 1.7.2–1.7.4 below or at the start of the relevant chapter.

1.7.2 Latin upper case symbols

A_c	Total cross-sectional area of a concrete section
A_p	Area of a prestressing tendon or tendons
A_s	Area of reinforcement within the tension zone
A_{s2}	Area of reinforcement in the compression zone at the ultimate limit state
A_{sw}	Cross-sectional area of shear reinforcement
E_{cd}	Design value of the secant modulus of elasticity
$E_{c(t)}$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at time t
$E_c, E_{c(28)}$	Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at 28 days
E_{cm}	Secant modulus of elasticity of normal weight concrete
E_s	Modulus of elasticity of reinforcement or prestressing steel
I_c	Second moment of area of a concrete section
$J_{(t, t_0)}$	Creep function at time t
M_{Sd}	Design value of the applied internal bending moment
N_{Sd}	Design value of the applied axial force (tension or compression)
$P_{m,t}$	Mean value of the prestressing force at time t , at any point distance x along the member
P_o	Initial force at the active end of the tendon immediately after stressing
T_{Sd}	Design value of the applied torsional moment
V_{Sd}	Design value of the applied shear force at the ultimate limit state

1.7.3 Latin lower case symbols

$1/r$	Curvature at a particular section
b	Overall width of a cross-section, or actual flange width in a T or L beam
d	Effective depth of a cross-section
d_g	Largest nominal maximum aggregate size
b_w	Width of the web on T, I or L beams
f_c	Compressive strength of concrete (Figure 3.1)
f_{cd}	Design value of concrete cylinder compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{ctk}	Characteristic axial tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
f_p	Tensile strength of prestressing steel
f_{pk}	Characteristic tensile strength of prestressing steel
$f_{p0.1}$	0.1 % proof-stress of prestressing steel
$f_{p0.1k}$	Characteristic 0.1 % proof-stress of prestressing steel
f_t	Tensile strength of reinforcement

f_{tk}	Characteristic tensile strength of reinforcement
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reinforcement
f_{ywd}	Design yield strength of stirrups
h	Overall depth of a cross-section
l	Length; span
l_{eff}	Effective span of a beam
s	Spacing of stirrups
t	Time being considered
t_o	Time at initial loading of the concrete
u	Perimeter of concrete cross-section, having area A_c
x	Neutral axis depth
z	Lever arm of internal forces

1.7.4 Greek symbols

γ_A	Partial safety factors for accidental actions A
γ_C	Partial safety factors for concrete material properties
γ_F	Partial safety factors for actions, F
γ_G	Partial safety factors for permanent actions G
γ_M	Partial safety factors for a material property, taking account of uncertainties in the material property itself and in the design model used
γ_P	Partial safety factors for actions associated with prestressing, P
γ_Q	Partial safety factors for variable actions Q
γ_s	Partial safety factors for the properties of reinforcement or prestressing steel
γ_f	Partial safety factors for actions without taking account of model uncertainties
γ_g	Partial safety factors for permanent actions without taking account of model uncertainties
γ_m	Partial safety factors for a material property, taking account only of uncertainties in the material property
ϵ_c	Compressive strain in the concrete
ϵ_{cl}	Compressive strain in the concrete at the peak stress f_c
ϵ_{cu}	Ultimate compressive strain in the concrete
ϵ_u	Elongation of reinforcement or prestressing steel at maximum load
ϵ_{uk}	Characteristic uniform elongation of reinforcement or prestressing steel at maximum load
μ	Coefficient of friction between the tendons and their ducts
ρ	Oven-dry density of concrete in Kg/m^3
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_w	Reinforcement ratio for shear reinforcement
σ_c	Compressive stress in the concrete

σ_{cu}	Compressive stress in the concrete at the ultimate compressive strain ϵ_{cu}
$\phi_{(t,t_0)}$	Creep coefficient, defining creep between times t and t_0 , related to elastic deformation at 28 days
ϕ	Diameter of a reinforcing bar or of a prestressing duct
ϕ_n	Equivalent diameter of a bundle of reinforcing bars
$\phi(\infty, t_0)$	Final value of creep coefficient

2 Basis of design

2.0 Notation — sections 2.1–2.4 (See also 1.6 and 1.7)

C_d	Nominal value of, or a function of certain design properties of materials
D_d	Design value of the damage indicator (fatigue)
$E_{d,dst}$	Design effects of destabilising actions
$E_{d,stab}$	Design effects of stabilizing actions
$G_{d,inf}$	Lower design value of a permanent action
$G_{d,sup}$	Upper design value of a permanent action
G_{IND}	Indirect permanent action
$G_{k,inf}$	Lower characteristic value of a permanent action
$G_{k,sup}$	Upper characteristic value of a permanent action
$G_{k,j}$	Characteristic values of permanent actions
Q_{IND}	Indirect variable action
$Q_{k,1}$	Characteristic value of one of the variable actions
$Q_{k,i}$	Characteristic values of the other variable actions
a_d	Design values of geometrical data
a_{nom}	Nominal value of geometrical data
Δ_a	Change made to nominal geometrical data for particular design purposes (e.g. assessment of effects of imperfections)
$\gamma_{G,inf}$	Partial safety factor for permanent actions, in calculating the lower design values
$\gamma_{G,sup}$	Partial safety factor for permanent actions, in calculating the upper design values
$\gamma_{GA}, \gamma_{GA,j}$	Partial safety factors for permanent actions, for accidental design situations
$\gamma_{G,j}$	Partial safety factor for any permanent action j
$\gamma_{Q,i}$	Partial safety factors for any variable action i
$\gamma_{Q,1}$	Partial safety factor for the basic most unfavourable variable action

2.1 Fundamental requirements

P(1) A structure shall be designed and constructed in such a way that

- with acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and its cost, and
- with appropriate degrees of reliability, it will sustain all actions and influences likely to occur during execution and use and have adequate durability in relation to maintenance costs.

P(2) A structure shall also be designed in such a way that it will not be damaged by events like explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.

(3) The potential damage should be limited or avoided by appropriate choice of one or more of the following:

- avoiding, eliminating or reducing the hazards which the structure is to sustain
- selecting a structural form which has low sensitivity to the hazards considered
- selecting a structural form and design that can survive adequately the accidental removal of an individual element
- tying the structure together.

P(4) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, design, construction and use as relevant to the particular project.

2.2 Definitions and classifications

2.2.1 Limit states and design situations

2.2.1.1 Limit states

P(1) **Limit states.** Limit states are states beyond which the structure no longer satisfies the design performance requirements.

Limit states are classified into:

- ultimate limit states
- serviceability limit states.

P(2) Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people.

P(3) States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also treated as ultimate limit states.

(4) Ultimate limit states which may require consideration include:

- loss of equilibrium of the structure or any part of it, considered as a rigid body.
- failure by excessive deformation, rupture, or loss of stability of the structure or any part of it, including supports and foundations.

see 4.2, 4.3.

P(5) Serviceability limit states correspond to states beyond which specified service requirements are no longer met.

(6) Serviceability limit states which may require consideration include:

- deformations or deflections which affect the appearance or effective use of the structure (including the malfunction of machines or services) or cause damage to finishes or non-structural elements
- vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness
- cracking of the concrete which is likely to affect appearance, durability or water tightness adversely
- damaging of concrete in the presence of excessive compression which is likely to lead to loss of durability.

see 4.4.1, 4.4.2, 4.4.3.

2.2.1.2 Design situations

P(1) Design situations are classified as:

- persistent situations corresponding to normal conditions of use of the structure
- transient situations, for example during construction or repair
- accidental situations

2.2.2 Actions

2.2.2.1 Definitions and principal classifications⁵⁾

P(1) An action (F) is:

- a force (load) applied to the structure (direct action), or
- an imposed deformation (indirect action); for example, temperature effects or settlement.

P(2) Actions are classified:

- i) by their variation in time
 - permanent actions (G), e.g. self-weight of structures, fittings, ancillaries and fixed equipment
 - variable actions (Q), e.g. imposed loads, wind loads or snow loads
 - accidental actions (A), e.g. explosions or impact from vehicles
- ii) by their spatial variation
 - fixed actions, e.g. self-weight [but see 2.3.2.3(2) for structures very sensitive to variations in self-weight].
 - free actions, which result in different arrangements of actions, e.g. movable imposed loads, wind loads, snow loads.

(3) Prestressing (P) is a permanent action but, for practical reasons, it is treated separately (see 2.5.4).

(4) Indirect actions are either permanent G_{IND} (e.g. settlement of support) or variable Q_{IND} (e.g. temperature) and are treated accordingly.

P(5) Supplementary classifications relating to the response of the structure are given in the relevant clauses.

2.2.2.2 Characteristic values of actions

P(1) Characteristic values F_k are specified

- in Eurocode 1 or other relevant loading codes, or
- by the client, or the designer in consultation with the client, provided that minimum provisions, specified in the relevant codes or by the competent authority, are observed.

P(2) For permanent actions where the coefficient of variation is large or where the actions are likely to vary during the life of the structure (e.g. for some superimposed permanent loads), two characteristic values are distinguished, an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$). Elsewhere a single characteristic value (G_k) is sufficient.

(3) The self-weight of the structure may, in most cases, be calculated on the basis of the nominal dimensions and mean unit masses.

P(4) For variable actions the characteristic value (Q_k) corresponds to either:

- the upper value with an intended probability of not being exceeded, or the lower value with an intended probability of not being reached, during some reference period, having regard to the intended life of the structure or the assumed duration of the design situation, or
- the specified value.

P(5) For accidental actions the characteristic value A_k (when relevant) generally corresponds to a specified value.

⁵⁾ Fuller definitions of the classifications of actions will be found in the Eurocode 1, Bases of Design and Actions on Structures.

2.2.2.3 Representative values of variable actions⁶⁾

P(1) The main representative value is the characteristic value Q_k .

P(2) Other representative values are expressed in terms of the characteristic value Q_k by means of a factor Ψ_i . These values are defined as

- combination value : $\Psi_0 Q_k$
- frequent value : $\Psi_1 Q_k$
- quasi-permanent value: $\Psi_2 Q_k$

P(3) Supplementary representative values are used for fatigue verification and dynamic analysis.

P(4) The factors Ψ_i are specified

- in Eurocode 1 or other relevant loading codes, or
- by the client or the designer in conjunction with the client, provided that minimum provisions, specified in the relevant codes or by the competent public authority, are observed.

2.2.2.4 Design values of actions

P(1) The design value F_d of an action is expressed in general terms as

$$F_d = \gamma_F F_k$$

P(2) Specific examples are:

$$G_d = \gamma_G G_k$$

$$Q_d = \gamma_Q Q_k \text{ or } \gamma_Q \Psi_i Q_k \quad (2.1)$$

$$A_d = \gamma_A A_k \text{ (if } A_d \text{ is not directly specified)}$$

$$P_d = \gamma_P P_k$$

where

γ_F , γ_G , γ_Q , γ_A and γ_P are the partial safety factors for the action considered taking account of, for example, the possibility of unfavourable deviations of the actions, the possibility of inaccurate modelling of the actions, uncertainties in the assessment of effects of actions, and uncertainties in the assessment of the limit state considered.

P(3) The upper and lower design values of permanent actions are expressed as follows [see 2.2.2.2 (P2)]:

- where only a single characteristic value G_k is used, then:

$$G_{d,\text{sup}} = \gamma_{G,\text{sup}} G_k$$

$$G_{d,\text{inf}} = \gamma_{G,\text{inf}} G_k$$

- where upper and lower characteristic values of permanent actions are used, then:

$$G_{d,\text{sup}} = \gamma_{G,\text{sup}} G_{k,\text{sup}}$$

$$G_{d,\text{inf}} = \gamma_{G,\text{inf}} G_{k,\text{inf}}$$

where $G_{k,\text{sup}}$ and $G_{k,\text{inf}}$ are the upper and lower characteristic values of permanent actions

and $\gamma_{G,\text{sup}}$ and $\gamma_{G,\text{inf}}$ are the upper and lower values of the partial safety factor for the permanent actions

⁶⁾ Fuller definitions of the classifications of actions will be found in the Eurocode 1, Bases of Design and Actions on Structures.

2.2.2.5 Design values of the effects of actions

P(1) The effects of actions (E) are responses (for example internal forces and moments, stresses, strains) of the structure to the actions. Design values of the effects of actions (E_d) are determined from the design values of the actions, geometrical data and material properties when relevant:

$$E_d = E(F_d, a_d, \dots) \quad [2.2(a)]$$

where a_d is defined in 2.2.4

(2) In some cases, in particular for non linear analysis, the effect of the randomness of the intensity of the actions and the uncertainty associated with the analytical procedures, e.g. the models used in the calculations, should be considered separately. This may be achieved by the application of a coefficient of model uncertainty, either applied to the actions or to the internal forces and moments.

(3) One possible procedure, called “linearization procedure”, may be schematically represented by the following equation:

$$E_d = \gamma_{S_d} E(\gamma_g G_k, \gamma_q Q_k \dots) \quad [2.2(b)]$$

and involves making a non-linear analysis until the level $\gamma_g G_k, \gamma_q Q_k \dots$ and then increasing E by the application of the factor γ_{S_d} .

2.2.3 Material properties

2.2.3.1 Characteristic values

P(1) A material property is represented by a characteristic value X_k which in general corresponds to a fractile in the assumed statistical distribution of the particular property of the material, specified by relevant standards and tested under specified conditions.

P(2) In certain cases a nominal value is used as the characteristic value.

(3) A material strength may have two characteristic values, an upper and a lower. In most cases only the lower value will need to be considered. In some cases, different values may be adopted depending on the type of problem considered. Where an upper estimate of strength is required (e.g. for the tensile strength of concrete for the calculation of the effects of indirect actions) a nominal high value of the strength may have to be established.

(4) The approach in P(1) above does not apply to fatigue.

2.2.3.2

design values

P(1) The design value X_d of a material property is generally defined as

$$X_d = \frac{X_k}{\gamma_M} \quad (2.3)$$

where

γ_M is the partial safety factor for the material property, given in 2.3.3.2 and 2.3.4.

In 4.3.5, other definitions apply.

P(2) Design values for the material properties, geometrical data and effects of actions, when relevant, shall be used to determine the design resistance R_d from:

$$R_d = R(X_d, a_d, \dots) \quad (2.4)$$

(3) The design value R_d may be determined from tests. Guidance is given in

2.2.4 Geometrical data

P(1) Design values of geometrical data describing the structure are generally represented by their nominal values:

$$a_d = a_{nom} \quad [2.5(a)]$$

P(2) In some cases the geometrical design values are defined by:

$$a_d = a_{nom} + \Delta a \quad [2.5(b)]$$

The values of Δa are given in the appropriate clauses.

(3) For imperfections to be adopted in the global analysis of the structure, see sections 2.5.1.3 and 4.3.5.4.

2.2.5 Load arrangements and load cases⁷⁾

P(1) A load arrangement identifies the position, magnitude and direction of a free action.

P(2) A load case identifies compatible load arrangements, sets of deformations and imperfections considered for a particular verification.

2.3 Design requirements

2.3.1 General

P(1) It shall be verified that no relevant limit state is exceeded.

P(2) All relevant design situations and load cases shall be considered

P(3) Possible deviations from the assumed directions or positions of actions shall be considered.

P(4) Calculations shall be performed using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. The models shall be sufficiently precise to predict the structural behaviour, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

2.3.2 Ultimate limit states

2.3.2.1 Verification conditions

P(1) When considering a limit state of static equilibrium or of gross displacements or deformations of the structure, it shall be verified that,

$$E_{d,dst} < E_{d,stb} \quad [2.6(a)]$$

where $E_{d,dst}$ and $E_{d,stb}$ are the design effects of destabilizing and stabilizing actions, respectively.

P(2) When considering a limit state of rupture or excessive deformation of a section, member or connection (fatigue excluded) it shall be verified that:

$$S_d \leq R_d \quad [2.6(b)]$$

where S_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments) and R_d is the corresponding design resistance, associating all structural properties with the respective design values (see 2.5.3).

P(3) When considering a limit state of transformation of the structure into a mechanism, it shall be verified that a mechanism does not occur unless actions exceed their design values — associating all structural properties with the respective design values.

P(4) When considering a limit state of stability induced by second-order effects it shall be verified that instability does not occur unless actions exceed their design values — associating all structural properties with the respective design values. In addition, sections shall be verified according to P(2) above.

P(5) When considering a limit state of rupture induced by fatigue it shall be verified that

$$D_d \leq 1 \quad [2.6(c)]$$

where D_d is the design value of the damage indicator: see Part E.

2.3.2.2 Combinations of actions

P(1) For each load case, design values E_d for the effects of actions shall be determined from combination rules involving design values of actions as identified by Table 2.1.

⁷⁾ Detailed rules on load arrangements and load cases are given in will be found in the Eurocode 1, Bases of Design and Actions on Structures.

Table 2.1 — Design values for actions for use in the combination of actions

Design situation	Permanent actions G_d	Variable Actions		Accidental actions A_d
		One with its characteristic value	Others with their combination value	
Persistent and Transient	$\gamma_G G_k$	$\gamma_Q Q_k$	$\Psi_0 \gamma_Q Q_k$	—
Accidental ^a	$\gamma_{GA} G_k$	$\Psi_1 Q_k$	$\Psi_2 Q_k$	$\gamma_A A_k$ (if A_d is not specified directly)

^a If not specified differently elsewhere.

P(2) The design values of Table 2.1 shall be combined using the following expressions (given in symbolic form)⁸⁾

— Persistent and transient design situations for verifications other than those relating to fatigue or prestressing (fundamental combinations):

$$\sum \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad [2.7(a)]$$

— Accidental design situations (if not specified differently elsewhere)

$$\sum \gamma_{GA,j} G_{k,j} + A_d + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i} \quad [2.7(b)]$$

where

$G_{k,j}$	characteristic values of permanent actions
$Q_{k,1}$	characteristic value of one of the variable actions
$Q_{k,i}$	characteristic value of the other variable actions
A_d	design value (specified value) of the accidental action
$\gamma_{G,J}$	partial safety factors for permanent action J
$\gamma_{GA,J}$	as $\gamma_{G,J}$, but for accidental design situations
$\gamma_{Q,i}$	partial safety factors for variable action i
Ψ_0, Ψ_1, Ψ_2	coefficients defined in 2.2.2.3

Imposed deformations should be considered where relevant.

P(3) Combinations for accidental design situations either involve an explicit accidental action A (e.g. shock) or refer to a situation after an accidental event ($A = 0$). Unless specified otherwise, $\gamma_{GA} = 1$ may be used.

P(4) In the expressions [2.7(a)] and [2.7(b)] prestressing shall be introduced where relevant.

(5) For fatigue, see Part E.

(6) Simplified equations for building structures are given in 2.3.3.1.

(7) For seismic design, see Eurocode 8.

(8) For fire design, see Eurocode 2-10.

2.3.2.3 Design values of permanent actions

P(1) In the various combinations defined above, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values, those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values [see 2.2.2.4(3)].

⁸⁾ Fuller definitions of the classifications of actions will be found in the Eurocode 1, Bases of Design and Actions on Structures.

P(2) Except for the case in P(3) below, either the lower or the upper design value (whichever gives the more unfavourable effect) shall be applied throughout the structure.

P(3) Where the results of a verification may be very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and the favourable parts of this action shall be considered as individual actions. This applies in particular to the verification of static equilibrium. In the afore-mentioned cases specific γ_G values need to be considered [see 2.3.3.1(3) for building structures].

(4) For continuous beams without cantilevers the same design value of the self-weight [evaluated as in 2.2.2.2(3)] may be applied to all spans.

2.3.3 Partial safety factors for ultimate limit states

2.3.3.1 Partial safety factors for actions on building structures

(1) Partial safety factors for the persistent and transient design situations are given in Table 2.2.

(2) For accidental design situations to which expression [2.7(b)] applies, the partial safety factors for variable actions and prestressing are equal to |unity|.

Table 2.2 — Partial safety factors for actions in building structures for persistent and transient design situations

	Permanent actions (γ_G)	Variable actions (γ_Q)		Prestressing (γ_p)
		One with its characteristic value	Others with their combination value	
Favourable effect	<u>1.0</u> ^a	<u>—</u> ^b	^b	^c <u>0.9</u> or <u>1.0</u>
Unfavourable effect	<u>1.35</u> ^a	<u>1.5</u>	<u>1.5</u>	^c <u>1.2</u> or <u>1.0</u>

^a See also paragraph (3) below
^b See Eurocode 1; in normal cases for building structures $\gamma_{Q,inf} = 0$.
^c See relevant clauses

(3) Where, according to 2.3.2.3 P(3), favourable and unfavourable parts of a permanent action need to be considered as individual actions, the favourable part should be associated with $\gamma_{G,inf} = |0.9|$ and the unfavourable part with $\gamma_{G,sup} = |1.1|$

P(4) Prestressing. For the evaluation of local effects (anchorage zones, bursting pressure) an effect equivalent to the ultimate characteristic strength shall be applied to the tendons (see 2.5.4).

(5) For the verification of the design of prestressed elements, the γ_p values in Table 2.2 should generally be used. However, for the evaluation of the combined effects of prestressing and of self-weight, reduced values of partial safety factors, which do not include allowances for analytical uncertainty, may be used (e.g. $\gamma_p = |1.0|$ and $\gamma_G = |1.2|$), where prestressing is favourable).

(6) Imposed deformations. Where non-linear methods of analysis are used the factors for variable actions given above apply. For a linear calculation, the factor for unfavourable effects shall be reduced by |20 %| (i.e. $\gamma_Q = 1.2$).

(7) Vectorial effects. Where components of a vectorial force act independently, the factors applied to any favourable component shall be reduced by |20 %|.

(8) By adopting the γ values given in Table 2.2, the expression [2.7(a)] may be replaced by the following:

— for design situations with only one variable action Q_{k1}

$$\Sigma \gamma_{G,j} G_{k,j} + |1.5| Q_{k,1} \quad [2.8(a)]$$

— for design situations with two or more variable actions Q_{k1}

$$\Sigma \gamma_{G,j} G_{k,j} + \frac{|1.35|}{1} \sum_{i=1}^n Q_{k,i} \quad [2.8(b)]$$

whichever gives the most unfavourable effects.

2.3.3.2 Partial safety factors for materials

(1) Partial safety factors for material properties are given in Table 2.3.

Table 2.3 — Partial safety factors for material properties

Combination	Concrete γ_c	Steel reinforcement or prestressing tendons γ_s
Fundamental	1.5	1.15
Accidental (except earthquakes)	1.3	1.0

(2) These values are assumed to take account of differences between the strength of test specimens of the structural materials and their strength in situ.

(3) The values given above are valid when the quality control procedures given in chapter 7 are followed. They apply to characteristic values defined in chapter 3 and for design data as described in chapter 4.2.

(4) Higher or lower values of γ_c may be used if these are justified by adequate control procedures.

(5) These values do not apply for fatigue verification.

(6) Where structural properties are determined by testing, see the relevant Part of this Code.

2.3.4 Serviceability limit states

P(1) It shall be verified that

$$E_d \leq C_d \text{ or } E_d \leq R_d$$

where

C_d is a nominal value or a function of certain design properties of materials related to the design effects of actions considered, and

E_d is the design effect of actions, determined on the basis of one of the combinations defined below.

The required combination is identified in the particular clause for serviceability verification (see 4.4).

P(2) Three combinations of actions for serviceability limit states are defined by the following expressions:

Rare combination

$$\sum G_{k,j} (+P) + Q_{k,1} + \sum_{i > 1} \psi_{0,i} Q_{k,i} \quad [2.9(a)]$$

Frequent combination

$$\sum G_{k,j} (+P) + \psi_{1,1} Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i} \quad [2.9(b)]$$

Quasi-permanent combination

$$\sum G_{k,j} (+P) + \sum_{i \geq 1} \psi_{2,i} Q_{k,i} \quad [2.9(c)]$$

where the notation is defined in 2.3.2.2(2).

Imposed deformations should be considered when relevant.

(3) Upper limits of compressive stresses in the concrete in the presence of rare and quasi permanent combinations may be fixed to avoid damage to the concrete and excessive creep deformations (see chapter 4.4.1.).

(4) An upper limit of the tensile stress in the steel may be fixed to reduce the risk of inelastic deformation and permanently open cracks (see chapter 4.4.1.).

P(5) Where simplified compliance rules are given in the relevant clauses dealing with serviceability limit states, detailed calculations using combinations of actions are not required.

P(6) Where the design considers the compliance of serviceability limit states by detailed calculations, simplified expressions may be used for building structures.

(7) For building structures the rare combination may be simplified to the following expressions, which may also be used as a substitute for the frequent combination.

— design situations with only one variable action, $Q_{k,1}$

$$\Sigma G_{k,j} (+P) + Q_{k,1} \quad [2.9(d)]$$

— design situations with two or more variable actions, $Q_{k,i}$

$$\Sigma G_{k,j} (+P) + 0.9 \Sigma_{i \geq 1} Q_{k,i} \quad [2.9(e)]$$

whichever gives the larger value.

P(8) Values of γ_M shall be taken as 1.0, except where stated otherwise in particular clauses.

2.4 Durability

P(1) To ensure an adequately durable structure, the following inter-related factors shall be considered:

- the use of the structure;
- the required performance criteria;
- the expected environmental conditions;
- the composition, properties and performance of the materials;
- the shape of members and the structural detailing;
- the quality of workmanship, and level of control;
- the particular protective measures;
- the likely maintenance during the intended life.

P(2) The environmental conditions shall be estimated at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials.

2.5 Analysis

2.5.1 General provisions

2.5.1.0 Notation (see also 1.6 and 1.7)

H_{fd}	Additional horizontal force to be considered in the design of horizontal structural elements, when taking account of imperfections
ΔH_j	Increase in the horizontal force acting on the floor of a frame structure, due to imperfections
$N_{ba}\}$ $N_{bc}\}$ $\}$	Design axial forces on columns or walls adjacent to a horizontal load transferring element, when considering imperfections
1	Total height of a structure in metres
n	Number of vertical continuous members acting together
α_n	Reduction coefficient in calculating v (Equation 2.11)
v	Angle of inclination of a structure, assumed in assessing the effects of imperfections

2.5.1.1 General

P(1) The purpose of analysis is the establishment of the distribution of either internal forces and moments, or stresses strains and displacements, over the whole or part of a structure. Additional local analysis shall be carried out where necessary.

(2) In most normal cases analysis will be used to establish the distribution of internal forces and moments; however, for certain complex elements, the methods of analysis used (e.g. finite element analysis) give stresses, strains and displacements rather than internal forces and moments. Special methods are required to use these results to obtain appropriate reinforcement areas.

P(3) Analyses are carried out using idealisations of both the geometry and the behaviour of the structure. The idealisations selected shall be appropriate to the problem being considered.

(4) The geometry is commonly idealised by considering the structure to be made up of linear elements, two dimensional elements and, occasionally, shells. Geometrical idealisations are considered in **2.5.2**.

(5) Common behavioural idealisations used for analysis are:

- elastic behaviour (see **2.5.3.2–2.5.3.3**)
- elastic behaviour with limited redistribution (see **2.5.3.4.2**)
- plastic behaviour (see **2.5.3.5.5**), including strut and tie models (see **2.5.3.7**)
- non-linear behaviour (see Appendix 2)

(6) Additional local analyses may be necessary where the assumption of linear strain distribution does not apply, e.g.

- supports
- under concentrated loads
- beam and beam-column intersections
- anchorage zones
- changes in section.

2.5.1.2 Load cases and combinations

P(1) For the relevant combinations of actions, sufficient load cases shall be considered to enable the critical design conditions to be established at all sections within the structure or part of the structure considered.

(2) Depending on the type of structure, its function or the method of construction, design may be carried out primarily for either the ultimate limit state or the serviceability limit. In many cases, provided that checks for one of these limit states have been carried out, checks for the other may be dispensed with as compliance can be seen by experience.

(3) Simplified combinations of actions and load cases may be used, if based on a reasonable interpretation of the structural response.

(4) For continuous beams and slabs in buildings without cantilevers subjected to dominantly uniformly distributed loads, it will generally be sufficient to consider only the following load cases (see **2.3.2.2**).

- a) alternate spans carrying the design variable and permanent load ($\gamma_Q Q_k + \gamma_G G_k$), other spans carrying only the design permanent load, $\gamma_G G_k$.
- b) any two adjacent spans carrying the design variable and permanent loads ($\gamma_Q Q_k + \gamma_G G_k$). All other spans carrying only the design permanent load, $\gamma_G G_k$.

(5) For linear elements and slabs in buildings, the effects of shear and longitudinal forces on the deformations may be ignored where these are likely to be less than 10 % of those due to bending.

2.5.1.3 Imperfections

P(1) In the ultimate limit state, consideration shall be given to the effects of possible imperfections in the geometry of the unloaded structure. Where significant, any possible unfavourable effect of such imperfections shall be taken into account.

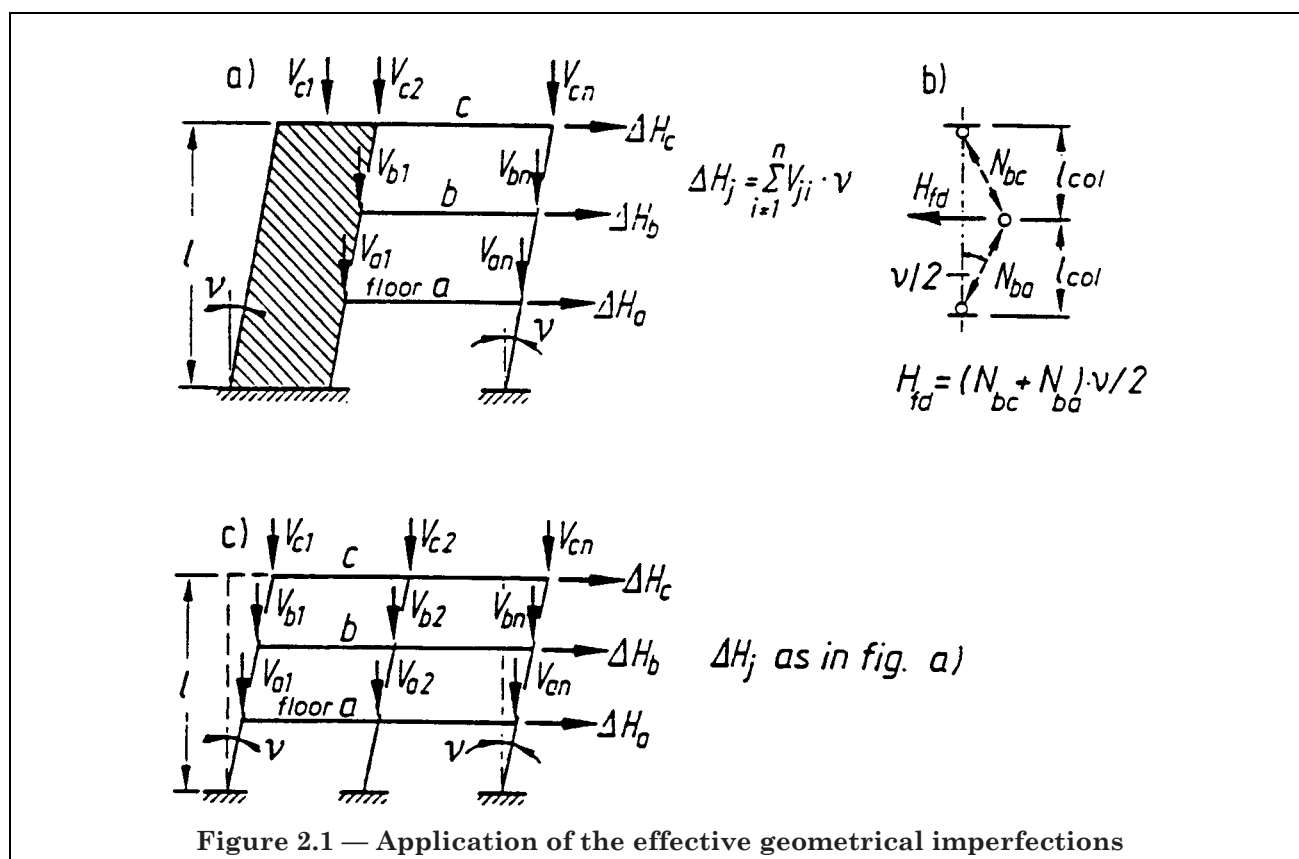
P(2) Individual sections shall be designed for the internal forces and moments arising from global analysis, combining effects of actions and imperfections of the structure as a whole.

(3) In the absence of other provisions, the influence of structural imperfections may be assessed by representing them as an effective geometrical imperfection using a procedure such as that given in (4) to (8) below.

(4) Where a structure is being analysed as a whole, the possible effects of imperfections may be assessed by assuming that the structure is inclined at an angle v to the vertical where:

$$v = 1/(100\sqrt{l}) \text{ (radians)} \quad (2.10)$$

where l is the total height of the structure in metres. v should not be taken as less than $|1/400|$ for cases where second order effects are insignificant or $|1/200|$ where they have to be taken into account (e.g. 4.3.5.4). [see Figure 2.1 a), Figure 2.1 b) and Figure 2.1 c)].



a) imperfections for the calculation of the horizontal forces on the bracing element

b) imperfections for the calculation of the horizontal forces in the floors transferring stabilising forces from the braced sub-assembly to the bracing elements.

c) equivalent horizontal forces ΔH acting on an unbraced frame

(5) For cases where n vertical elements act together, v , given by (4) above may be reduced by the factor α_n given by Equation (2.11)

$$\alpha_n = \sqrt{(1 + 1/n)/2} \quad (2.11)$$

In Figure 2.1 a), n is $n = 2$; in Figure 2.1 c), n is $n = 3$.

(6) If more convenient, the deviations from the vertical given by (4) above may be replaced by equivalent horizontal forces which should be taken into account in the overall analysis of the structure, bracing elements, supports and ties (see Figure 2.1 a), Figure 2.1 b) and Figure 2.1 c).

(7) Structural elements which are assumed to transfer stabilising forces from the elements of a structure to be braced to the bracing elements should be designed to carry an additional horizontal force H_{fd} [see Figure 2.1 b)] such that:

$$H_{fd} = (N_{bc} + N_{ba}) v/2 \quad (2.12)$$

where:

N_{bc} and N_{ba} denote the design axial forces on the adjacent columns or walls, acting on the load transferring element being considered

H_{fd} should not be taken into account in the design of the bracing elements.

(8) Where the effects of imperfections are smaller than the effects of design horizontal actions, their influence may be ignored. Imperfections need not be considered in accidental combinations of actions.

2.5.1.4 Second order effects

P(1) Second order effects shall be taken into account where they may significantly affect the overall stability of a structure or the attainment of the ultimate limit state at critical sections.

(2) For normal buildings, second order effects may be neglected where they increase the moments, calculated ignoring displacements, by not more than 10 %.

2.5.1.5 Time dependent effects

P(1) Time dependent effects shall be taken into account where significant.

(2) Creep and shrinkage normally need only be considered for the serviceability limit state except where their influence in the ultimate limit state are likely to be significant.

2.5.1.6 Design by testing

P(1) The design of structures or structural elements may be based on testing.

(2) In this case, a specification for the test programme and interpretation of the results should be approved nationally.

2.5.2 Idealisation of the structure

2.5.2.0 Notation (See also 1.6 and 1.7)

a_1, a_2, a_i	Coefficients used in calculating values for effective spans (Equation 2.15 and Figure 2.4)
b_{eff}	Effective flange width of a T or L beam
h_f	Overall depth of a flange in T or L beams
l_{eff}	Effective span of beams and slabs
l_n	Clear distance between the faces of the supports
l_o	Length of span(s) between points of zero moment
t	Thickness of a supporting element

2.5.2.1 Structural models for overall analysis

P(1) The elements of a structure are normally classified, by consideration of their nature and function, as beams, columns, slabs, walls, plates, arches, shells etc. Rules are provided for the analysis of the commoner of these elements and of structures consisting of combinations of these elements.

(2) To be considered as a beam or column, the span or length of the member should not be less than twice the overall section depth. A beam whose span is less than twice its depth is considered as a deep beam.

(3) To be considered as a slab, the minimum span should not be less than four times the overall slab thickness.

(4) A slab subjected to dominantly uniformly distributed loads may be considered to be one-way spanning if either:

- a) it possesses two free (unsupported) and sensibly parallel edges or
- b) if it is the central part of a sensibly rectangular slab supported on four edges with a ratio of the longer to shorter span greater than 2.

(5) Ribbed or waffle slabs may be treated as solid slabs for the purposes of analysis, provided that the flange or structural topping and transverse ribs have sufficient torsional stiffness. This may be assumed provided that:

- the rib spacing does not exceed 1 500 mm

- the depth of the rib below the flange does not exceed four times its width
 - the depth of the flange is at least 1/10 of the clear distance between ribs or 50 mm, whichever is the greater.
 - transverse ribs are provided at a clear spacing not exceeding $\lfloor 10 \rfloor$ times the overall depth of the slab.
- The minimum flange thickness of 50 mm may be reduced to 40 mm where permanent blocks are incorporated between the ribs.

(6) A wall should have a horizontal length of at least four times its thickness. Otherwise it should be treated as a column.

2.5.2.2 Geometrical data

2.5.2.2.1 Effective width of flanges (all limit states)

P(1) In T beams the effective flange width depends on the web and flange dimensions, the type of loading, the span, the support conditions and the transverse reinforcement.

(2) For analysis, when a great accuracy is not required (e.g. continuous beams in buildings), a constant width may be assumed over the whole span.

(3) The effective width for a symmetrical T beam may be taken as:

$$b_{\text{eff}} = b_w + \frac{1}{5} l_o < b \quad (2.13)$$

and, for an edge beam (i.e. with flange on one side only)

$$b_{\text{eff}i} = b_w + \frac{1}{10} l_o \leq b_i + b_w \quad (i = 1 \text{ or } 2) \quad (2.14)$$

(for the notations see Figure 2.2 and Figure 2.3 below).

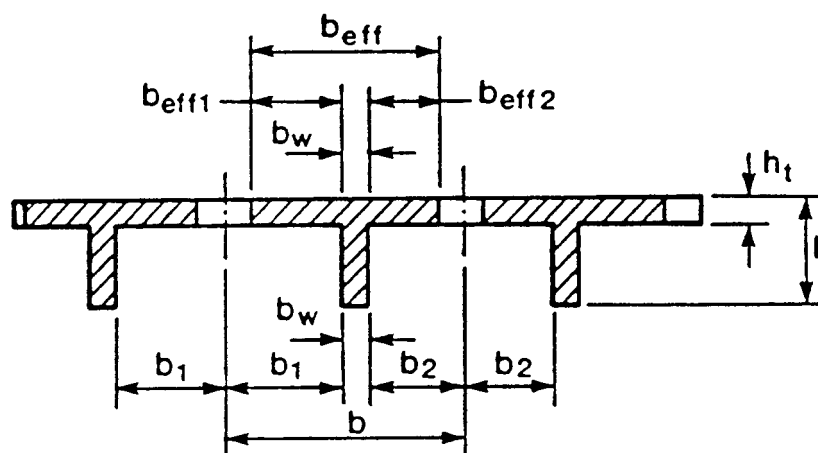


Figure 2.2 — Definition of dimensions

(4) The distance l_o between points of zero moment may be obtained from Figure 2.3 for typical cases.

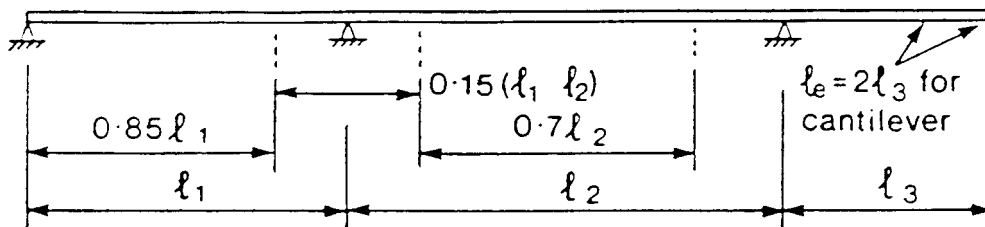


Figure 2.3 — Approximate effective spans for calculation of effective breadth ratios

The following conditions should be satisfied:

- i) The length of the cantilever should be less than half the adjacent span.
 - ii) The ratio of adjacent spans should lie between 1 and 1.5.
- (5) For the dispersion of prestressing forces in T beams see 4.2.3.5.3.

2.5.2.2.2 Effective span of beams and slabs

(1) The effective span (l_{eff}) of a member may be calculated as follows:

$$l_{\text{eff}} = l_n + a_1 + a_2 \quad (2.15)$$

where:

l_n is the clear distance between the faces of the supports

values for a_1 and a_2 , at each end of the span, may be determined from the appropriate a_i values in Figure 2.4.