DD ENV 1992-2:2001

Eurocode 2: Design of concrete structures

Part 2. Concrete bridges

(together with United Kingdom National Application Document)

ICS 91.080.40; 93.040

Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/10, Bridges, upon which the following bodies were represented:

Association of Consulting Engineers

British Cement Association

British Construction Steelwork Association Ltd.

British Precast Concrete Federation Ltd.

British Railway Board

British Waterways Board

County Surveyors' Society

Department of the Environment Transport and the Regions (Highways Agency)

Institution of Civil Engineers

Institution of Structural Engineers

Steel Construction Institute

UK Steel Association

Welding Institute

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Contents

National foreword

This Draft for Development was prepared by Subcommittee B/525/10 and is the English language version of ENV 1992-2:1996 *Eurocode 2: Design of concrete structures — Part 2: Concrete bridges*, as published by the European Committee for Standardization (CEN). This Draft for Development also includes the United Kingdom (UK) National Application Document (NAD) to be used with the ENV in the design of buildings to be constructed in the UK.

ENV 1992-2 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structural and geotechnical design of building and civil engineering works.

This publication should not be regarded as a British Standard.

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 publication of this ENV and its National Application Document should be considered to supersede any reference to a British Standard in previous DD ENV Eurocodes concerning the subject covered by these documents.

The values for certain parameters in the ENV Eurocodes may be set by individual 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 of the ENV.

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 the Secretary of Subcommittee B/525/10, BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, a proposed revision within two years of the issue of this document.

This document does not puport to include all the necessary provisions of a contract. Users of this document are responsible for its correct application.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to xxiv, the ENV title page, pages 2 to 45 and a back cover.

The BSI copyright notice displayed in this document indicates when the document was last issued.

National Application Document

for use in the UK with ENV 1992-2:1996

Contents of National Application Document

Introduction

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

a) a textual examination of ENV 1992-2:1996 and ENV 1992-1-1:1991, ENV 1992-1-3:1994, ENV 1992-1-4:1994, ENV 1992-1-5:1994 and ENV 1992-1-6:1994;

b) a parametric calibration examination against BS 5400-4, supporting standards and test data;

c) trial calculations.

1 Scope

This NAD provides information to enable ENV 1992-2:1996 (hereafter referred to as EC2-2) to be used with ENV 1992-1-1:1991, ENV 1992-1-3:1994, ENV 1992-1-4:1994, ENV 1992-1-5:1994 and ENV 1992-1-6:1994, as qualified by their respective NADs, for the design and construction of bridges in the UK.

2 Normative references

The following normative documents contain provisions, which, through reference in this text, constitute provisions of this National Application Document. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest editions of the publication referred to applies.

Standards publications

BS 5400-4, *Steel, concrete and composite bridges — Code of practice for design of concrete bridges.*

ENV 1991-3:1994, *Eurocode 1: Basis of design and actions on structures — Part 3: Traffic loads on bridges.*

ENV 1992-1-1:1991, *Eurocode 2: Design of concrete structures — Part 1-1: General rules and rules for buildings.*

ENV 1992-1-3:1994, *Eurocode 2: Design of concrete structures — Part 1-3: General rules — Precast concrete elements and structures.*

ENV 1992-1-4:1994, *Eurocode 2: Design of concrete structures — Part 1-4: General rules — Lightweight aggregate concrete with closed structure.*

ENV 1992-1-5:1994, *Eurocode 2: Design of concrete structures — Part 1-5: Unbonded and external prestressing tendons.*

ENV 1992-1-6:1994, *Eurocode 2: Design of concrete structures — Part 1-6: General rules — Plain concrete structures.*

Other documents

GREAT BRITAIN. HIGHWAYS AGENCY. *Design manual for roads and bridges. Vol. 1. Highway structures: approval procedures and general design — Section 3: General design — Loads for highway bridges.* Publication no. BD 37/88. London: The Stationery Office, 1994.

GREAT BRITAIN. HIGHWAYS AGENCY. *Manual of Contract Documents for Highway Works — Volume 1: Specification for Highway Works*. London: The Stationery Office, 1998.

3 Partial factors, combination factors and other values

The partial factors, combination factors and other values are as follows.

a) The values for combination factors (ψ) should be those given in Table 3 and Table 4 of the NAD for use with ENV 1991-3:1994.

b) The values for partial factors should be those given in EC2-2, except as modified by the UK NADs to the various parts of ENV 1992-1-1.

c) ENV 1992-1-1:1991, **2.5.3.5.5** (5) should not be modified as indicated in Table 3 of its NAD.

d) Other values should be those given in EC2-2 except for those given in Table 1 of this NAD.

Table 1 — Values to be used in referenced clauses instead of boxed values

Reference in ENV 1992-1-1	Reference in ENV 1992-1-3	Reference in ENV 1992-2	Definition	UK values
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	Δh = 5 mm
4.1.3.3 (8)			Allowance for tolerance Δh in cover for in situ concrete	Δh = 5 mm
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	2
4.3.2.4.4 (1)			Limit to cotθ in the variable strut inclination method for beams with constant reinforcement	$0.5 < \cot\theta < 2.0$
4.3.3.1 (6)			Limits to cotθ in torsion calculation	$0.5 < \cot\theta < 2.0$
4.3.4.5.2 (1)			V_{rd2}	2.0 [see also **6.3**i) of this NAD]
		4.3.7.5 (101)	Fatigue stress range	See **6.3**c) of this NAD
		4.4.2.2.1 (103)	Maximum bar spacing	300
	4.5.2 (103)		Limit to average bearing stress	$0.8f_{cd}$
Table 5.1			Minimum diameter of mandrels	See Table 7 of this NAD

Table 1 — Values to be used in referenced clauses instead of boxed values (*continued*)

Reference in ENV 1992-1-1	Reference in ENV 1992-1-3	Reference in ENV 1992-2	Definition	UK values
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	300 mm
5.4.3.3 (2)			Minimum shear as a percentage of the total for beams	100 %
6.2.2 (1)			Tolerances	See **5.5** of this NAD

4 Loading documents

The loading documents to be used are:

ENV 1991-3 for traffic loads;
BD 37/88 for all other loads.

5 Reference standards

Standards including materials specifications and standards for construction are listed for reference in Table 2a), Table 2b) and Table 2c) of this NAD.

Table 2a) — References — References in ENV 1992-2 to other publications

Reference in ENV 1992-2	Document referred to	Document title or subject area	UK document	Highways Agency document
1.1.2 P(104)	Eurocode 1	Basis of design and actions on structures	BS 5400-1 BS 5400-2[a]	BD 15/92 [1] BD 37/88
1.1.2 P(104)	Eurocode 8	Design of structures in seismic regions	—	—
1.1.2 (105)	Eurocode 1-1	Basis of design	BS 5400-1	BD 15/92 [1]
1.1.2 (105)	Eurocode 1-2-1	Actions on structures: densities, self-weight and imposed loads	BS 5400-2[a]	BD 37/88
1.1.2 (105)	Eurocode 1-2-4	Actions on structures: wind actions	BS 5400-2[a]	BD 37/88
1.1.2 (105)	Eurocode 1-2-5	Actions on structures: thermal actions	BS 5400-2[a]	BD 37/88
1.1.2 (105)	Eurocode 2-1-2	Structural fire design	—	—
1.1.2 (105)	Eurocode 2-1-4	The use of lightweight aggregate concrete	BS 5400-4	BD 24/92 [2]
1.1.2 (105)	Eurocode 2-1-6	Plain concrete	BS 5400-4	BD 24/92 [2]
1.1.2 (105)	Eurocode 2-3	Concrete foundations	BS 8004	BD 32/88 [3]
1.1.2 (105)	Eurocode 7-1	Geotechnical design	BS 1377 BS 8004 BS 5930	BD 32/88 [3] BD 30/87 [4]
1.1.2 (105)	Eurocode 8-2	Earthquake resistant design of structures	—	—
1.4.1 P(104)	ENV 1991-1	Basis of design	BS 5400-1	BD 15/92 [1]
A107	prEN 10138	Prestressing steel	BS 4486 BS 5896	*Specification for Highway Works*
A107.1 (103)	ENV 1992-3	Types of stay cable	—	—
A107.4 (107)	ISO 161-1, ISO 3607[b] or other relevant standards	Specification for high density polyethylene (HDPE)	—	—

[a] This has been partially replaced by BS 5400-9.1:1983 and BS 5400-9.2:1983.
[b] This has now been replaced by ISO 11922-1 and ISO 11922-2.

Table 2b) — References — References in ENV 1992-1-1 to other publications

Reference in ENV 1992-1-1	Document referred to	Document title or subject area	UK document	Highways Agency document
1.1.1 P(4)	Eurocode 8	Design of structures in seismic regions	—	—
1.1.1 P(5)	Eurocode 1	Basis of design and actions on structures	BS 5400-1 BS 5400-2[a]	BD 15/92 [1] BD 37/88
3.2, **3.3** and **3.4**	ENV 10080 and relevant standards prEN 10138 and relevant standards	Reinforcing steel Prestressing steel	BS 4449 BS 4482 BS 4483 BS 4486 BS 5896	— —
3.4	Relevant standards European Approval Documents	Anchorages	BS 4447	—
4.1.2.3 (3)	ISO/DP 9690 ENV 206	Classification of environmental conditions for concrete structures	—	—
4.2.3.4.1	Relevant standards	Relaxation of prestressing steel	BS 4486 BS 5896	—
6.3.2.2	Appropriate national or international documents	Specification of finishes	BS 5400-7	*Specification for Highway Works*
6.3.3.1 P(1)	Relevant Euronorms or CEN, ISO or national standards, National Building Regulations Control Authority	Requirements for reinforcing steel	BS 4449 BS 4482 BS 4483 BS 5400-7	*Specification for Highway Works*
6.3.3.2 P(3)	Appropriate international or national standards	Cutting and bending of reinforcement	BS 4466	*Specification for Highway Works*
6.3.3.3 P(3)	International or national standards	Welding of reinforcement	BS 7123	BA 40/93 [5] *Specification for Highway Works*
6.3.3.3 P(4)	Relevant standards	Fatigue requirements for welding of reinforcement	BS 5400-7 BS 5400-10	BD 9/81 [6], BA 40/93 [5]
6.3.3.3 P(5)	International or national standards	Production and checking of welded connections	BS 7123	BA 40/93 *Specification for Highway Works*
6.3.3.4 (3)	Standards or approval documents	Mechanical connectors	BS 5400-4	BD 24/92 *Specification for Highway Works*

[a] This has been partially replaced by BS 5400-9.1:1983 and BS 5400-9.2:1983.

Table 2b) — References — References in ENV 1992-1-1 to other publications (*continued*)

Reference in ENV 1992-1-1	Document referred to	Document title or subject area	UK document	Highways Agency document
6.3.4.1	Relevant Euronorms or CEN, ISO or national standards, National Building Regulations Control Authority	Requirements for prestressing steel	BS 4486 BS 5896	*Specification for Highway Works*
6.3.4.3	Standards or approval documents	Devices for jointing, anchorage and coupling of tendons	BS 4447	—
6.3.4.6.2 P(4)	EN 447	Types of cement for grouting	Concrete Society Technical Report No. 47 [7]	—
7.5	CEC or National Administrative Procedures	Control of design	—	BD 2/89 [8]
7.6.5	Relevant technical documents	All other structural materials	—	*Specification for Highway Works*

Table 2c) — References — References in ENV 1992-1-3 for precast concrete bridges to other publications

Reference in ENV 1992-1-3	Document referred to	Document title or subject area	UK document	Highways Agency document
6.2.1 (104)	Relevant CEN product standards	Tolerances of construction and workmanship	BS 5400-7	*Specification for Highway Works*
6.3.5 (101)	Relevant CEN product and other standards	Construction and workmanship of precast elements and structures	BS 5400-4 BS 5400-7	BD 24/92 *Specification for Highway Works*

6 Additional recommendations

6.1 Chapter 1. Introduction

a) *Clause **1.1.2** P(101)*

All references to ENV 1992-1-1 in EC2-2 shall be interpreted as being to ENV 1992-1-1 as qualified by its UK NAD.

b) *Clause **1.1.2** (105)*

All references to any ENV shall be interpreted as being to that ENV as qualified by its UK NAD.

6.2 Chapter 3. Material properties

a) *Clause **2.5.4.2** (4)*

Clause **2.5.4.2** (104) of ENV 1992-1-3:1994 is applicable only to pretensioning.

6.3 Chapter 4. Section and member design

a) *Clause* ***4.1.3.3*** *(6)*

For the relevant exposure class defined in Table 3 of this NAD, the nominal concrete cover to all reinforcement including links and stirrups should not be less than the appropriate values given in Table 4 of this NAD. However, for pretensioned precast units the values in Table 4 of this NAD should be reduced by 10 mm. Where de-icing agents are used in a region, structures shall be classified as exposure Class 3 unless it can be guaranteed that the type of de-icing agent to which the structure will be exposed will have no deleterious effect on the reinforcement. Bridges over non-electrified railways shall be classified as exposure Class 5b and the nominal cover to reinforcing bars should not be less than 45 mm.

Table 3 — Exposure classes related to environmental conditions

Exposure class		Environment	Examples
1 Moderate		Concrete surfaces above ground level and fully sheltered against all of the following: — rain; — de-icing salts; — sea water spray. Concrete surfaces permanently saturated by water with a pH > 4.5.	Surfaces protected by waterproofing or by permanent formwork. Interior surface of pedestrian subways, voided superstructures or cellular abutments. Concrete permanently under water.
2 Severe	2a Without frost	Concrete surfaces exposed to driving rain. Concrete surfaces exposed to alternative wetting and drying.	Wall and structure supports remote from the carriageway. Bridge deck soffits. Buried parts of structures.
	2b With frost	As 2a but also exposed to freezing and thawing	As 2a.
3 Very severe		Concrete surfaces directly affected by de-icing salts.	Walls and structures within 10 m of the carriageway, parapet edge beams and buried structures less than 1 m below carriageway level.
4 Extreme	4a Without frost	Concrete surfaces in saturated salt air. Concrete surfaces exposed to abrasive action by sea water. Concrete surfaces exposed to water with a pH ≤ 4.5.	Concrete adjacent to the sea. Marine structures. Parts of structure in contact with moorland water.
	4b With frost	As 4a but also exposed to freezing and thawing.	As 4a above.
5 Aggressive[a]	5a	Concrete surfaces exposed to a slightly aggressive chemical environment.	Concrete in an aggressive industrial atmosphere. Parts of structure in contact with contaminated ground.
	5b	Concrete surfaces exposed to a moderately aggressive chemical environment.	Parts of structure in contact with contaminated ground.
	5c	Concrete surfaces exposed to a highly aggressive chemical environment.	Parts of structure in contact with contaminated ground.

[a] Chemically aggressive environments are classified in ISO/DP9690. The following equivalent exposure conditions may be assumed:
- Exposure class 5a: ISO classification A1G, A1L, A1S;
- Exposure class 5b: ISO classification A2G, A2L, A2S;
- Exposure class 5c: ISO classification A3G, A3L, A3S.

Table 4 — Nominal cover requirements for normal weight concrete[a]

Exposure class		Location	Nominal cover mm			
			Concrete grade			
			C20/25	C25/30	C30/37	C40/50 and above
1		3	55	45	40	35
2	2a	2	[b]	80	80	80
		3	[b]	55	45	40
	2b	2	[b]	80[a]	80	80
		3	[b]	55[a]	45	40
3		1	[b]	[c]	85[a]	85
		2	[b]	[c]	80[a]	80
		3	[b]	[c]	60[a]	50
4	4a	1	[b]	[b]	85	85
		2	[b]	[b]	80	80
		3	[b]	[b]	75	65
	4b	1	[b]	[b]	85[a]	85
		2	[b]	[b]	80[a]	80
		3	[b]	[b]	75[a]	65
5	5a 5b 5c		This exposure can occur alone or in combination with the above classes. In selecting an appropriate cover the designer should consider other relevant exposure classes, such as cement content, type of cement, water:cement ratio and the use of protective membranes.			

Location: 1 — tendons in slabs where the upper surface is directly exposed to de-icing agents (i.e. no protective membrane);
2 — cast against an earth face;
3 — other locations.

NOTE For pretensioned precast units the tabulated values may be reduced by 10 mm.

[a] Air entrained concrete should be specified.
[b] Concrete grade not permitted.
[c] Parapet beams only, nominal cover = 70 mm.

b) *Clause* ***4.2.1.3.3*** *(11)*

α should be taken as 0.85 for both short-term and long-term effects.

c) *Clause* ***4.2.3.5.6***

Where a pre-tensioned tendon or group of tendons is enclosed by transverse reinforcement with an area of at least 1 000 mm^2/m β_b may be taken as 50 % of the appropriate value given in Table 4.7 for all strand with areas up to 225 mm^2.

d) *Clause* ***4.3.2.2*** *(11)*

In addition:

d) in the case of a pile cap, enhancement should be applied only to those portions of the section where the flexural reinforcement is fully anchored by passing across the head of a pile.

e) *Clause* ***4.3.2.3*** *(1)*

Equation (4.18)

Replace Equation (4.18) with:

$V_{Rd1} = \lfloor 0.21k\ (100\rho_1\ f_{ck})^{0.33} + 0.15\sigma_{cp} \rfloor\ b_w d$

Change the definitions as follows:

Delete:

τ_{Rd}

Add:

$k = (500/d)^{0.25})$

$\rho_1 = A_{s1}\ /\ b_w d\ \leq\ |0.03|$

A_{s1} = the area of tension reinforcement extending not less than d beyond the section considered

Other terms are as defined previously.

Delete Table 4.8.

f) *Clause* ***4.3.2.4.3*** *(1)*

The equation should be replaced as follows:

Equation (4.22):

$V_{Rd3} = V_{wd} + V_{cd} - 0.4$

g) *Clause* ***4.3.2.5*** *(4)*

Delete:

τ_{Rd} is taken from Table 4.8 in **4.8.2.3**

and insert:

τ_{Rd} is given in the following Table 4.8.

γ_c = 1.5 for different concrete strengths

f_{ck}	12.0	16.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
τ_{Rd}	0.18	0.22	0.26	0.30	0.34	0.37	0.41	0.44	0.48

h) *Clause* ***4.3.4.1*** *(9)*

Does not apply.

i) *Clause* ***4.3.4.2.1*** *(1)*

Items 1) and 2) should be replaced with:

1) In the case of a rectangular loaded area having a perimeter greater than 11d and/or a ratio of length to breadth greater than 2.0, the critical perimeter according to Figure 4.17 only should be taken into account, in the absence of a more detailed analysis.

j) *Clause **4.3.4.2.2** (1)*

If a part of a perimeter cannot, physically, extend 1.5d from the boundary of the loaded area, then the part perimeter shall be taken as far from the loaded areas as is physically possible and the value of V_{Rd1}, given in **4.3.4.5.1** (1), for that part may be increased by a factor 1.5d/x, where x is the distance from the boundary of the loaded area to the perimeter actually considered.

k) *Clause **4.3.4.5.1***

Replace Equation (4.56) and the definitions with the following:

$$V_{Rd1} = \lfloor 0.21k(100\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp} \rfloor b_w d \tag{4.56}$$

where

$k = (500/d)^{0.25}$;

$\rho_1 = \sqrt{\rho_{1x}\ \ \rho_{1y}}$;

ρ_{1x} ρ_{1y} are ratios in the x and y directions calculated for a width equal to the side dimension of the column (or loaded area) plus 3d to either side of it (or to slab edge if it is closer);

$d = (d_x + d_y)/2$;

d_x and d_y are the effective depths of the slab at the points of intersection between the design failure surface and the longitudinal reinforcement, in the x and y direction respectively.

l) *Clause **4.3.4.5.1** (2)*

The upper limit of 0.015 applies to $\sqrt{\rho_{1x}\ \ \rho_{1y}}$ and not to ρ_1.

m) *Clause **4.3.4.5.2** (1)*

In Equation (4.57), in addition to the limitation on V_{Rd2} given in Table 1 of this NAD the shear stress at the perimeter of the coloumn 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) should be used:

$$V_{Rd3} = 1.4V_{Rd1} + (0.3\sum A_{sw} f_{ya} \sin\alpha) / u \tag{4.58a}$$

n) *Clause **4.3.5.3.5** P(101)*

The effective height, l_o, of a column may be determined using Table 5 of this NAD where l_{col} is the clear height between end restraints.

The values given in Table 5 are based on the following assumptions:

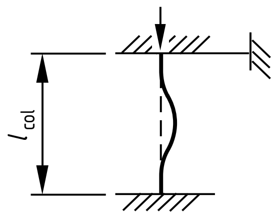
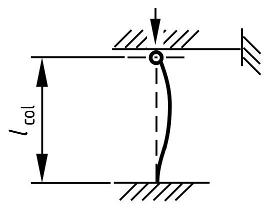
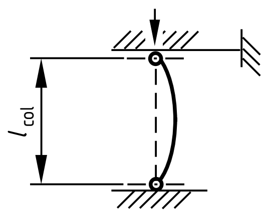
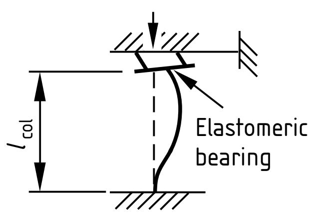
a) rotational restraint is at least $4E_{cm}I_{col}/l_{col}$ for cases 1, 2 and 4 to 6 and $8E_{cm}I_{col}/l_{col}$ for case 7;

b) lateral and rotational rigidity of elastomeric bearings are zero.

Where a more accurate evaluation of the effective height is required or where the end stiffness values are less than those values given in a), the effective heights should be derived from first principles.

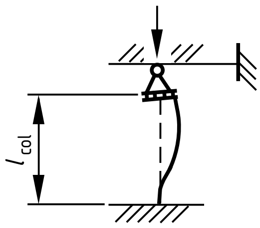
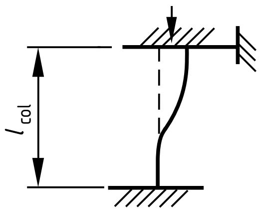
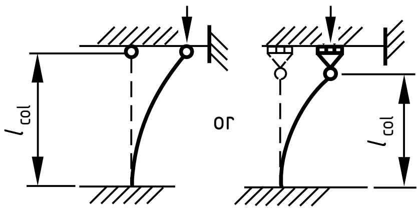
The accommodation of movements and the method of articulation chosen for the bridge will influence the degree of restraint developed for columns. These factors should be assessed as accurately as possible using engineering principles based on elastic theory and taking into account all relevant factors such as foundation flexibility, type of bearings, articulation system, etc.

Table 5 — Effective height, l_o, for columns

Case	Idealized column and buckling mode	Restraints			Effective height, l_c
		Location	Position	Rotation	
1		Top	Full	Full[a]	$0.70\ l_{col}$
		Bottom	Full	Full[a]	
2		Top	Full	None	$0.85\ l_{col}$
		Bottom	Full	Full[a]	
3		Top	Full	None	$1.0\ l_{col}$
		Bottom	Full	None	
4		Top	None[a]	None[a]	$1.3\ l_{col}$
		Bottom	Full	Full[a]	

[a] Assumed value [see **6.3**n)].

Table 5 — Effective height, l_o, for columns (*continued*)

Case	Idealized column and buckling mode	Restraints			Effective height, l_c
		Location	Position	Rotation	
5		Top	None	None	1.4 l_{col}
		Bottom	Full	Full[a]	
6		Top	None	Full[a]	1.5 l_{col}
		Bottom	Full	Full[a]	
7		Top	None	None	2.3 l_{col}
		Bottom	Full	Full[a]	

[a] Assumed value [see **6.3**n)].

o) *Clause **4.3.5.6***

Notwithstanding the references to buildings in clause **4.3.5.6**, it should be assumed that this clause is applicable also to bridge structures.

p) *Clause **4.3.5.6.4** (4)*

When Equation (4.72) is used to calculate the curvature *1/r*, then interaction of biaxial bending should be considered using:

$$\left(\frac{M_x}{M_{Rdx}}\right)^{\alpha_n} + \left(\frac{M_y}{M_{Rdy}}\right)^{\alpha_n} \leq 1.0$$

where

M_x and M_y	are the moments about the major *x–x* axis and minor *y–y* axis respectively due to ultimate loads;
M_{Rdx}	is the ultimate moment capacity about the major *x–x* axis assuming an ultimate axial load capacity, N_{ud}, not less than the value of the ultimate axial load, *N*;
M_{Rdy}	is the ultimate moment capacity about the minor *y–y* axis assuming an ultimate axial load capacity, N_{ud}, not less than the value of the ultimate axial load, *N*;

$\alpha_n = 0.667 + 1.67\ N/N_{ud} \geq 1.0$ and ≤ 2.0.

However, when the curvature is calculated using a non-linear analysis in each of the *x* and *y* directions, α_n may be assumed to be 1.0.

q) *Clause **4.3.5.7** (2)*

The second of Equations (4.77) should be replaced with:

$l_{ot} < 250\ b^2/d$

r) *Clause **4.3.7.1** (102)*

When applying b) to railway bridges the depth of ballast should not be included in assessing the depth of earth cover.

In addition to the situations listed in **4.3.7.1** (102), a fatigue verification for road bridges is not generally necessary for the local effects of wheel loads applied directly to a slab spanning between beams or webs provided that:

1) the clear span to overall depth ratio of the slab does not exceed 18;

2) the slab acts compositely with its supporting beams or webs;

3) either:

i) the slab also acts compositely with transverse diaphragms; or

ii) the width of the slab perpendicular to its span exceeds three times its clear span.

s) *Clause **4.3.7.5** (101)*

For road bridges, replace |70| N/mm² with the appropriate value from Table 6a) and Table 6b) of this NAD. It is emphasized that the fatigue resistance of welded bars shall be checked using **4.3.7.5** (102).

NOTE Table 6b) need only be applied to those slabs that do not conform to the criteria in **6.3**r) of this NAD.

Table 6a) — Limiting stress ranges (N/mm²) — Longitudinal bending for unwelded reinforcing bars in road bridges

Span m	Adjacent spans loaded		Alternate spans loaded	
	Bars ≤ 16 mm ϕ	Bars > 16 mm ϕ	Bars ≤ 16 mm ϕ	Bars > 16 mm ϕ
<3.5	150	115	210	160
3.5 to 5	125	95	175	135
5 to 10	110	85	175	135
10 to 20	110	85	140	110
20 to 100	90	70	110	85
100 to 200	115	90	135	105
>200	190	145	200	155

Table 6b) — Limiting stress ranges (N/mm²) — Transverse bending for unwelded reinforcing bars in road bridges

Span mm	Bars ≤ 16 mm ϕ	Bars > 16 mm ϕ
<3.5	210	160
3.5 to 5	120	90
5 to 10	70	55

t) *Clause **4.3.7.5** (105)*

In line 1 replace "P(103)" with "P(104)".

u) *Clause **4.3.7.6***

It is not necessary to apply **4.3.7.6**, if only prestressing steel is present at the section under consideration.

v) *Clause **4.3.7.8***

In Table 4.117, the values of $\Delta\sigma_{Rsk}$ for straight and bent bars of 195 N/mm² and 180 N/mm², respectively, should be reduced to 162 N/mm² and 150 N/mm², respectively, for bars with a diameter greater than 16 mm.

Welds in reinforcing steel, including tack welds, should not be used in bridges carrying rail traffic without prior approval of the relevant authority.

Welds in reinforcing steel should not be used in a deck slab spanning between longitudinal and/or transverse members and subjected to the effect of concentrated wheel loads in a traffic lane. Lap welding should not be used to connect reinforcing bars subjected to fatigue loading.

w) *Clause **4.4.0.3** (102)*

In the case of continuous bridges consisting of precast pretensioned beams with their ends embedded in in-situ concrete crossheads at the supports, verification criterion B should be adopted for the embedded lengths of the beams during the construction phase.

x) *Clause **4.4.2.1** P(109)*

Replace the existing clause with the following:

> "For design crack width, members prestressed with permanently unbonded tendons without bonded tendons may be treated as ordinary reinforced concrete members."

y) *Clause **4.4.2.2.1** P(101)*

Replace the existing clause with the following:

> "For reasons of durability and appearance of the concrete a minimum reinforcement area shall be provided in reinforced or prestressed bridge structures in order to prevent wide single cracks due to imposed deformations not considered in the design calculations, self-equilibrating stresses or distribution of prestress."

z) *Clause* ***4.4.2.2.2*** *(101)*

Replace the existing clause with the following:

"The minimum reinforcement area according to Equation (4.19.4) should be placed in sections where, under the infrequent combination of actions, the concrete stresses are tensile or less than 1 N/mm^2 compressive. For box girders and I-beams the web and flanges may be treated separately for this purpose."

aa) *Clause* ***4.4.2.2.3*** *(101)*

In the definition of ρ_p the words, "within an area of not more than 300 mm around the ordinary reinforcement" should be deleted.

bb) *Clause* ***4.4.2.2.3*** *(101)*

Replace definitions of A_{ct}, σ_s, k, N_{sd} with the following:

A_{ct} is the area of the tensile zone immediately prior to cracking of the cross section web or flange as appropriate taking the tensile strength of concrete as f_{ctm};

σ_s is the steel stress in the minimum reinforcement area according to Table 4.120, σ_s may be increased by a factor $\eta = (f_{ctm}/f^*_{ctm})^{1/2}$;

where

f^*_{ctm} = 2.5 N/mm^2;

and

f_{ctm} is the assumed mean tensile strength of concrete, σ_s should not exceed kf_{yk};

and

k is a coefficient which takes account of the effect of secondary crack formation which leads to a reduction of restraint forces;

k = 1.0 for webs or rectangular sections with $h \leq 0.3$ m or flanges with widths less than 0.3 m; and

= 0.65 for webs or rectangular sections with $h \geq 0.8$ m or flanges with widths greater than 0.8 m, intermediate values may be interpolated;

N_{sd} is the axial force (compression force negative) at the serviceability limit state acting on the part of the cross section under consideration. N_{sd} should be determined considering characteristic values of prestress and axial forces under quasi-permanent combinations of actions or the minimum axial force that can co-exist with the bending moment considered.

In the definition of k_c add the following at the end:

If $k_c \leq 0$, no reinforcing steel is required.

cc) *Clause* ***4.4.2.3*** *(103)*

Replace the existing clause with the following:

"In design cases according to (102) above, the crack width may be considered adequately controlled if either the bar diameter does not exceed the values given in Table 4.120 or the maximum bar spacing does not exceed the limit in Table 4.121".

In these tables, σ_s is the stress in the reinforcing steel unless there is prestressing steel alone, in which case σ_s is equal to $\Delta\sigma_p$.

The steel stress for the application of Tables 4.120 or 4.121 should be calculated under the relevant combination of actions using Equations (4.198) or (4.199), as appropriate.

dd) *Clause* ***4.4.2.3*** *(106)*

Replace the definition of σ''_s with the following:

"σ''_s is steel stress in the reinforcing steel or change of stress in prestresing steel relative to the stress state at decompression, calculated in the cracked state assuming full bond under the relevant combination of actions."

Add the following at end of definitions of ϕ_s and ϕ_p:

"However if there are a variety of sizes, ϕ shall be taken as the weighted average size ($4As/\Sigma\pi\phi$, that is 4 times total steel area over total steel perimeter)."

ee) *Clause* ***4.4.2.3***, *Table 4.120*

Add at the end of note at bottom "and the bar diameter shall be taken as the equivalent diameter of the tendon ϕ_p".

ff) *Clause* ***4.4.2.3***, *Tables 4.120*

Add "or $\Delta\sigma$p" after σ_{s}" in heading to the left-hand column.

gg) *Clause* ***4.4.2.3***

Replace the existing Table 4.121 with the following:

Table 4.121 — Maximum bar spacing for high bond bars

Maximum bar spacing (mm)			
Steel stress (bending) σ_s or $\Delta\sigma$p N/mm^2	**Pure flexure (reinforced sections)**	**Pure tension (reinforced sections)**	**Pre-stressed sections**
80	—	300	300
120	—	250	250
160	300	200	200
200	250	150	150
240	200	125	100
280	150	75	50
320	100	—	—
360	50	—	—

6.4 Chapter 5. Detailing provisions

a) *Table 5.1*

Table 5.1 should be replaced by Table 7 of this NAD, which gives minimum diameters of mandrels.

Table 7 — Minimum diameters of mandrels

	Hooks, bends, loops (see Figure 5.2 of ENV 1992-1-1)		**Bent-up bars or other curved bars**		
	Bar diameter		**Value of minimum concrete cover, perpendicular to plane of curvature**		
	ϕ<20 mm	$\phi \geq$ 20 mm	>100 mm and >7ϕ	>50 mm and >3ϕ	$\leq$ 50 mm and $\leq$ 3ϕ
Minimum diameter of mandrels for plain bars S 250	4ϕ	4ϕ	7ϕ	8.5ϕ	11.4ϕ
Minimum diameter of mandrels for high bond bars S 460	6ϕ	8ϕ	13ϕ	15.7ϕ	20.9ϕ

b) *Clause* ***5.2.6.3***

This clause does not apply to 40 mm diameter bars.

c) *Clause* ***5.4.3.2.3***

The additional recommendation in the NAD to EC2-1 is not appropriate when a full analysis (e.g. grillage or finite element) of a slab has been performed.

d) *Clause* ***5.4.3.3*** *(4)*

The ENV 1992-1-1:1991 NAD Additional Requirement **6.5**f) does not apply.

e) *Clause* ***5.4.8.1*** *(3)*

Replace Equation [5.22] with:

$$F_{\mathrm{Rdu}} = A_{\mathrm{co}}\,\alpha\,f_{\mathrm{cd}}\,\sqrt{(A_{\mathrm{c1}}/A_{\mathrm{co}})}\ \le 3.3\,\alpha\,f_{\mathrm{cd}}\,A_{\mathrm{co}}$$

where

α is as defined in ENV 1992-1-1:1991, **4.2.1.3.3** (11).

f) *Clause* ***5.4.9.3.3*** *(102)*

In the case of continuous or integral bridges consisting of precast pretensioned beams designed for verification criteria B (see **4.4.0.3**) with their ends embedded in in situ concrete at the supports, creep and shrinkage calculations are not required other than for estimating prestress losses provided the following apply:

1) either:

i) the angle of skew is not greater than 20°; or

ii) the angle of skew is less than 40° and the aspect ratio is not less than 1 where the aspect ratio is defined as the ratio of skew span to breadth normal to the skew span;

2) the area of longitudinal bottom steel per beam at the supports is not less than the minimum given by Equation (**4.194**) with f_{ctm} equal to the tensile strength of the interface between precast and in situ concrete which may be assumed to be 50 % of the tensile strength of the in situ concrete;

3) the area of steel distributed in b) is also not less than:

i) 3 000/s mm^2 for interior supports in bridges with three or more spans;

ii) 4 000/s mm^2 for the central support of a two span bridge;

iii) 1 500/s mm^2 at end supports in integral bridges;

where s is the beam spacing in metres but not less than 1;

4) where the live load analysis is done using uncracked section properties throughout, including for the reinforced in situ concrete support section, allowance is made in the serviceability analysis of the beams in sagging for the effect of a 10 % reduction in the support moment due to redistribution.

6.5 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 the *Manual of Contract Documents for Highway Works — Volume 1: Specification for Highway Works*, clauses **1710**, **1714**, **1715** and **1723**.

b) *Clause* ***6.3.3.3***

Additional guidance is given in the *Manual of Contract Documents for Highway Works — Volume 1: Specification for Highway Works*, clause **1717**.

6.6 Appendix 106. Damage equivalent stresses for fatigue verification

a) *Clause* ***A106.3.1*** *(103)*

The use of the values of $\lambda_{\mathrm{s,1}}$ given in Table A106.2 should be agreed with the relevant authority.

b) *Clause* ***A106.3.2*** *P(101)*

In the definitions of $S_{\mathrm{cd,min.equ}}$ and $S_{\mathrm{cd,max,equ}}$, replace Sd with γ_{Sd}.

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

EN 447:1996, *Grout for prestressing tendons — Specification for common grout.*

prEN 10138 (all parts), *Prestressing steel.*

ENV 206:1990, *Concrete — Performance, production, placing and compliance criteria.*

ENV 1991 (all parts), *Eurocode 1: Basis of design and actions on structures.*

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ENV 1992-1-2:1995, *Eurocode 2: Design of concrete structures — Part 1-2: General rules — Structural fire design.*

ENV 1992-3:1998, *Eurocode 2: Design of concrete structures — Part 3: Concrete foundations.*

ENV 1997-1:1994, *Eurocode 7: Geotechnical design — Part 1: General rules.*

ENV 1998 (all parts), *Eurocode 8: Design provisions for earthquake resistance of structures.*

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ISO/DP 9690, *Classification of environmental conditions for concrete structures.*

Other documents

[1] GREAT BRITAIN. HIGHWAYS AGENCY. *Design manual for roads and bridges — Vol. 1: Highway structures: approval procedures and general design — Section 3: General design — Part 2: General principles for the design and construction of bridges: use of BS 5400-1:1988.* Publication no. BD 15/92. London: The Stationery Office.

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[3] GREAT BRITAIN. HIGHWAYS AGENCY. *Design manual for roads and bridges — Vol. 2: Highway structures: design (substructures and special structures), materials —Section 1: Substructures — Piled foundations.* Publication no. BD 32/88. London: The Stationery Office.

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[8] GREAT BRITAIN. HIGHWAYS AGENCY. *Design manual for roads and bridges — Vol. 1: Highway structures — Approval procedures and general design — Section 1: Approval procedures and general design — Technical approval of highway structures on motorways and other trunk roads — Part 1: General procedure.* Publication no. BD 2/89. London: The Stationery Office.

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EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

EUROPÄISCHE VORNORM

ENV 1992-2

September 1996

ICS 93.040

Descriptors: civil engineering, concrete structures, bridges, design, building codes, computation

English version

Eurocode 2: Design of concrete structures - Part 2: Concrete bridges

Eurocode 2: Calcul des structures en béton - Partie 2: Ponts en béton

Eurocode 2: Planung von Stahlbeton- und Spannbetontragwerken - Teil 2: Betonbrücken

This European Prestandard (ENV) was approved by CEN on 1995-09-28 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 an European Standard (EN).

CEN members are required to announce the existance 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

Ref. No. ENV 1992-2:1996 E

Contents

Normative appendices

Informative appendix

Foreword

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 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.

(3) Until the necessary set of harmonized technical specifications for products and for the methods of testing their performances are available, some of the Structural Eurocodes cover some of these aspects in informative Annexes.

Background of the Eurocode programme

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized 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".

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

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

Eurocode programme

(7) 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 action 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 of structures for earthquake resistance
EN 1999	Eurocode 9	Design of aluminium alloy structures

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

(9) This Part 2 of Eurocode 2 is being published as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental applications and for the submission of comments.

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

(12) Meanwhile, feedback and comments on this Prestandard should be sent to the Secretariat of CEN/-TC250/SC2 at the following address:

Deutsches Institut für Normung e.V. (DIN)
Burggrafenstrasse 6
D - 10787 Berlin
phone: (+49) 30 - 26 01 - 25 01
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or to your national Standards Organisation.

National Application Documents (NAD'S)

(13) In view of the responsibilities of authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by [ ] ("boxed values"). The authorities in each member country are expected to assign definitive values to these safety elements.

(14) Some of the supporting European or International standards may not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document 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.

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

Matters specific to this Prestandard

(16) The scope of Eurocode 2 is defined in 1.1.1. of ENV 1992-1-1 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 of ENV 1992-1-1; these will cover additional technologies of applications, and will complement and supplement this Part.

(17) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions and conditions given in 1.3 of ENV 1992-1-1.

(18) The seven chapters in ENV 1992-1-1 are complemented by four Appendices. Some of the more detailed Principles / Application Rules, which are needed in particular cases, have been moved out of the main part of the text for clarity.

(19) As indicated in paragraph (14) 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 Prestandard 206 (Concrete - performance, production, placing and compliance criteria), and the durability requirements given in 4.1 of this Prestandard.

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

For ENV 1992-2, the following additional subclauses apply:

(21) This Part 2 of Eurocode 2 complements ENV 1992-1-1 for the particular aspects of concrete bridges.

(22) The framework and structure of this Part 2 correspond to ENV 1992-1-1. However, Part 2 contains Principles and Application Rules which are specific to concrete bridges.

(23) Where a particular subclause of ENV 1992-1-1 is not mentioned in this ENV 1992-2, that subclause of ENV 1992-1-1 applies as far as deemed appropriate in each case.
Some Principles and Application Rules in ENV 1992-1-1 are modified or replaced in this Part 2, in which case they are superseded.
Where a Principle or Application Rule in ENV 1992-1-1 is modified or replaced, the new number is identified by the addition of 100 to the original number. Where a new Principle or Application Rule is added, it is identified by a number which follows the last number of ENV 1992-1-1 with 100 added to it.
A subject not covered by ENV 1992-1-1 is introduced in this Part by a new subclause. The subclause number for this follows the most appropriate clause number in ENV 1992-1-1.

(24) The numbering of equations, figures, footnotes and tables in this Part follow the same principles as the clause numbering in (23) above.

(25) The scope of this Part 2 of Eurocode 2 is defined in 1.1.2 below. Additional Parts of Eurocode 2 and other Standards or Prestandards to which reference is made are also indicated in 1.1.2.

(26) For the application of this Part 2 of Eurocode 2 it is assumed that the relevant authorities or clients:
- define the load model and the characteristic values of the traffic loads according to ENV 1991-3 ;
- define, with regard to the environmental conditions of exposure, the verification criteria for the serviceability limit states; see 4.4.0.3 of this Part 2 for a possible classification.

1 Introduction

1.1 Scope

1.1.2 Scope of Part 2 of Eurocode 2

Replacement of this subclause in ENV 1992-1-1 by:

P(101) Part 2 of Eurocode 2 provides rules for the structural design of concrete bridges, which apply complementary to Parts 1-1, 1-3 and 1-5 of Eurocode 2. Unless stated otherwise in this Part 2, the Principles and Application Rules in these other Parts are applicable to bridges.

P(102) This Part 2 refers to road bridges, footbridges and railway bridges, the structure of which is made of reinforced and/or prestressed normal weight concrete. In addition, this Part 2 can be used for high strength concrete and lightweight aggregate concrete provided that the rules for these materials can be justified.

P(103) This Part 2 does not provide rules specific to other structural forms covered in the other Parts of Eurocode 2. In addition, when considering the effects on the structural design of structural bearings, barriers and other bridge equipment, reference shall be made to other relevant documents or specifications for the particular project (e.g., for road bridges some detailing rules concerning barriers and parapets are given in ENV 1991-3).

P(104) The engineering rules related to wind, earthquake and, if relevant, to other actions, given in Eurocode 1 and Eurocode 8 for bridges are applicable.

(105) When using this Part 2, where relevant, reference should be made to the following European Prestandards:

ENV 1991-1-1	Eurocode 1 Part 1-1	Basis of design
ENV 1991-2-1	Eurocode 1 Part 2-1	Densities, self-weight and imposed loads
ENV 1991-2-4	Eurocode 1 Part 2-4	Wind actions
ENV 1991-2-5	Eurocode 1 Part 2-5	Thermal actions
ENV 1991-3	Eurocode 1 Part 3	Traffic loads on bridges
ENV 1992-1-2	Eurocode 2 Part 1-2	Structural Fire Design
ENV 1992-1-3	Eurocode 2 Part 1-3	Precast concrete structures
ENV 1992-1-4	Eurocode 2 Part 1-4	Lightweight concrete
ENV 1992-1-5	Eurocode 2 Part 1-5	Unbonded and external prestressing tendons
ENV 1992-1-6	Eurocode 2 Part 1-6	Plain concrete
ENV 1992-3	Eurocode 2 Part 3	Concrete foundations
ENV 1997-1	Eurocode 7 Part 1	Geotechnical design
ENV 1998-2	Eurocode 8 Part 2	Earthquake resistance-Part: Bridges

1.2 Distinction between principles and application rules

Paragraphs P(1) to P(6) are replaced by the following :

(101) Depending on the character of the individual clauses, distinction is made in this Eurocode between principles and application rules.

(102) The principles comprise :

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

(103) The principles are identified by a letter P preceding the paragraph number in brackets.

(104) The application rules are generally recognised rules which follow the principles and satisfy their requirements. 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.

(105) In this Part, the application rules are identified by a number in brackets, as in this paragraph.

1.4 Definitions

1.4.1 Terms common for all Eurocodes

Addition after Principle P(3):

P(104) In addition to ISO 8930, the terminology used in ENV 1991-1 and ENV 1991-3 applies.

P(105) The following terms are used in common for all Eurocodes dealing with bridges (in brackets the French and German translations):

bridge (pont, Brücke) : civil engineering construction works mainly intended to carry loads related to communication over a natural obstacle or a communication line. This includes all types of bridges, especially road bridges, footbridges, railway bridges, canal bridges, airplane bridges.

abutment (culée, Widerlager) : any end support of a bridge without rigid continuity with the deck. Rigid abutments and flexible abutments should be distinguished where relevant.

pier (pile, Pfeiler) : intermediate support of a bridge, situated under the deck.

bearing (appareil d'appui, Lager) : structural device located between the deck and an abutment or pier of the bridge and transferring loads of the deck to the abutment or pier and, where relevant, allowing displacements.

cable stay (hauban, Schrägseil) : tensioned element acting as adjustable passive suspender of a cross-linked frame formed by the deck, the pylons and the cable stays.

prestress (précontrainte, Vorspannung) : permanent effect due to controlled forces and/or controlled deformations imposed on a structure. Various types of prestress shall be distinguished from each other as relevant (for example prestress by tendons, prestress by imposed deformation at supports).

headroom (hauteur libre, lichte Höhe) : the free height available for traffic.

P(106) For the terminology used for the verification of fatigue, ENV 1993-1-1 applies.

1.4.2 Special terms used in Part 2 of Eurocode 2

Addition after Principle P(2):

P(103) **coupling joints** : joints at locations where tendons are coupled.

1.6 Symbols common to all Eurocodes

1.6.3 Greek lower case letters

Addition at the end of this subclause:

ψ_1' Infrequent combination value

1.7 Special symbols used in this Part 2 of Eurocode 2

1.7.3 Latin lower case symbols

Addition:

f_{pt} Guaranteed ultimate tensile strength of prestressing steel used for stay cables

2 Basis of design

2.2 Definitions and classifications

2.2.1 Limit states and design situations

2.2.1.1 Limit States

Replacement of Application Rule (4) by:

(104) 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.
- failure caused by fatigue.

See 4.2, 4.3.

2.2.2 Actions

2.2.2.2 Characteristic values of actions

Addition after Principle P(5):

(106) During execution design values of loads should be calculated allowing for the equipment in use and an additional variable and free load due to persons, equal to |1 kN/m²|, should be taken into account.

2.2.2.3 Representative values of variable actions

Replacement of Principle P(2) by:

P(102) 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 : $\psi_0 Q_k$
- infrequent value : $\psi_1' Q_k$
- frequent value : $\psi_1 Q_k$
- quasi-permanent value : $\psi_2 Q_k$

Replacement of Principle P(4) by:

P(104) Factors ψ_i applicable to some relevant actions are given in Eurocode 1. Values of factors ψ_i for actions not given in Eurocode 1 shall be selected with due regards to the physical characteristics of the action.

2.2.3 Material properties

2.2.3.1 Characteristic values

Replacement of Application Rule (4) by:

(104) The approach in P(1) applies also to the verification of fatigue.

2.3 Design requirements

2.3.2 Ultimate limit states

2.3.2.2 Combinations of actions

Replacement of this subclause by:

P(101) For road bridges, footbridges and railway bridges, the combinations of actions defined in ENV 1991-3 shall be used. For other types of bridges the combinations shall be specified from relevant documents. For verification of fatigue see 4.3.7.2 of this Part 2.

(102) In general it is not necessary to take climatic temperature effects into account for the ultimate limit states.

P(103) Settlements shall be taken into account if they cause action effects which are significant compared to those from the direct actions.

(104) If settlements are taken into account, appropriate estimate values of predicted settlements should be used.

2.3.3 Partial safety factors for ultimate limit states

2.3.3.1 Partial safety factors for actions on bridge structures

Replacement of this subclause by:

(101) Partial safety factors should be taken from ENV 1991-3.
If not stated otherwise, γ_P = |1,0| should be assumed for the effects of prestress.

(102) For verification of fatigue see 4.3.7.

(103) Where, according to 2.3.2.3 P(3) in ENV 1992-1-1, favourable and unfavourable parts of a permanent action need to be considered as individual actions, the favourable part should be associated with $\gamma_{G,inf}$ = |0,95| and the unfavourable part with $\gamma_{G,sup}$ = |1,05|.

(104) Prestressing. For the evaluation of local effects (anchorage zones, bursting stresses) an effect equivalent to the characteristic breaking load should be applied to the tendons (see 2.5.4 in ENV 1992-1-1).

2.3.3.2 Partial safety factors for materials

Replacement of Application Rule (5) by:

(105) These values apply for fatigue verification.

2.3.4 Serviceability limit states

Replacement of Application Rule (7) by:

(107) The simplified combinations of actions in 2.3.4 (7) of ENV 1992-1-1 do not apply to bridges.

Addition after Principle P(8):

P(109) The combinations of actions for serviceability limit states are defined in ENV 1991-3 and ENV 1992-1-1. For imposed deformations not covered by relevant standards appropriate estimate values shall be used.

2.5 Analysis

2.5.1 General provisions

2.5.1.2 Load cases and combinations

Replacement of Application Rule (4) by:

(104) 2.5.1.2 (4) in ENV 1992-1-1 does not apply to bridges.

2.5.1.3 Imperfections

Replacement of Application Rule (3) by:

(103) 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 as given in (4) of subclause 2.5.1.3 in ENV 1992-1-1.

Replacement of Application Rules (5) to (8) by:

(105) Application Rules (5) to (8) in 2.5.1.3 of ENV 1992-1-1 do not apply to bridges.

2.5.2 Idealisation of the structure

2.5.2.1 Structural models for overall analysis

Replacement of Application Rule (5) by:

(105) Application Rule (5) in 2.5.2.1 of ENV 1992-1-1 does not apply to bridges.

2.5.2.2 Geometrical data

2.5.2.2.1 Effective width of flanges

Replacement of Application Rules (2) to (5) by:

(102) For analysis, when a great accuracy is not required, a constant width may be assumed over the whole span.

(103) For the analysis of the internal forces and moments, the verification at the ultimate limit state, the verification of the limit state of vibration and the calculation of deformations at the serviceability limit state, the actual flange width may be used.

(104) For the verification of stresses and crack widths at the serviceability limit state, and the verification of fatigue, the effective width for a symmetrical T-beam may be taken as:

$$b_{eff} = b_w + \frac{1}{5} l_0 \leq b \qquad (2.113)$$

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

$$b_{eff} = b_w + \frac{1}{10} l_0 \leq b_1 \text{ (or } b_2) \qquad (2.114)$$

(for the notations see Figures 2.102 and 2.103 below).

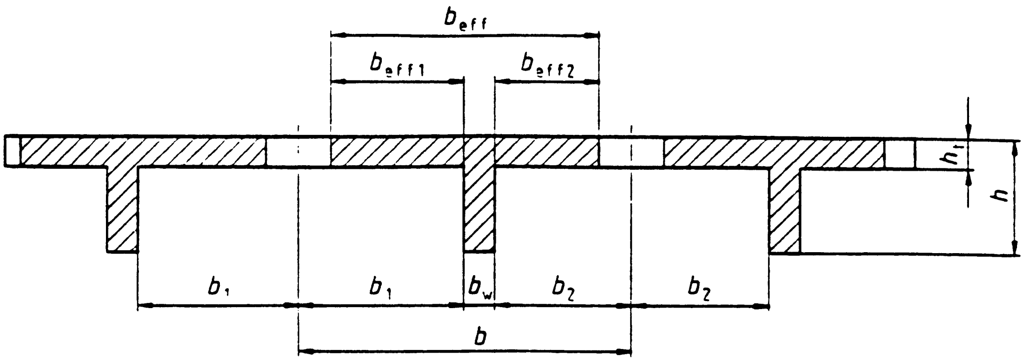
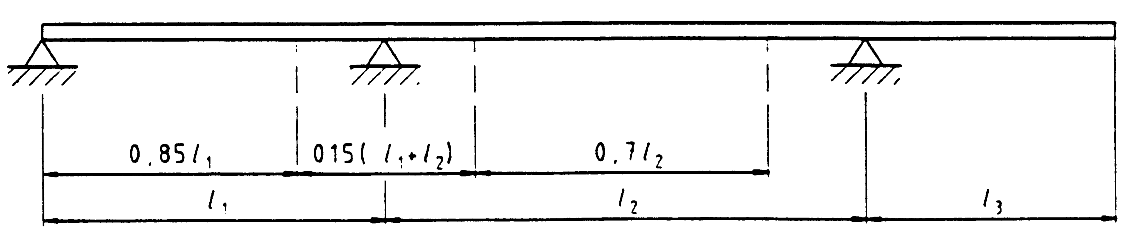


Figure 2.102: Definitions of dimensions of effective width of flanges

(105) The distance ℓ_0 between points of zero moment may be obtained from Figure 2.103 for typical cases.



$\ell_0 = 2\ \ell_3$ for cantilever

Figure 2.103: Approximate effective spans for calculation of effective width 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.

(106) For the dispersion of prestressing forces in T-beams see 4.2.3.5.3 in ENV 1992-1-1.

2.5.3 Calculation methods

2.5.3.1 Basic considerations

Replacement of Principle P(4) and Application Rule (5) by:

(104) Global analysis for imposed deformations due to temperature and shrinkage effects should be considered for the serviceability limit state, where relevant.

2.5.3.3 Simplifications

Replacement of Application Rules (2) by:

(102) The Application Rules (2), (3), (5) and (6) in ENV 1992-1-1 do not apply to bridges.

2.5.3.5 Analysis of slabs

2.5.3.5.1 Scope

Addition after Application Rule (3):

(104) For concentrated loads on bridge decks the theoretical loaded area used in the analysis should be taken as follows:

- for bending moments: according to chapter 4.3.2 of ENV 1991-3.
- for shear forces:an area limited by the critical perimeter given for punching shear according to 4.3.4.2 in ENV 1992-1-1.

3 Material properties

3.2 Reinforcing steel

3.2.2 Classification and geometry

Addition after Principle P(8):

(109) In general, only high ductile steel B500B according to ENV 10080 should be used for bridges.

4 Section and member design

4.1 Durability requirements

4.1.3 Design

4.1.3.3 Concrete cover

Addition after Application Rule (12):

(113) In general, the minimum concrete cover to a duct should not be less than |50mm|.
In the case of watertight concrete of class C40/50 (see ENV 206, Table 3 and 7.3.1.5) and above or concrete surfaces coated by impermeable adherent films, the minimum concrete cover to pretensioned tendons should not be less than |40mm|.
If tendons are placed under the surface of carriageway slabs or top slabs of footbridges and the surface is directly exposed to de-icing agents, the minimum cover to tendons and ducts should not be less than |80mm|.

(114) The minimum concrete cover to reinforcing steel should not be less than |30mm|.
Where the concrete surface is directly exposed to de-icing agents, saturated salt air, abrasive action by sea water or chemical environment (for example bridges over non-electrified railways) the minimum cover to reinforcing steel should not be less than |50mm|.

(115) In a highly aggressive chemical environment according to Table 4.1, Line 5c, in ENV 1992-1-1 a protective barrier should be provided to prevent direct contact with the aggressive media.

4.2 Design data

4.2.2 Reinforced concrete

4.2.2.3 Mechanical properties of reinforcing steel

4.2.2.3.3 Fatigue

Replacement of Application Rule (1) by:

(101) For fatigue verification see 4.3.7 of this Part 2.

4.2.3 Prestressed concrete

4.2.3.3 Mechanical properties of prestressing steel

4.2.3.3.5 Fatigue

Replacement of Application Rule (1) by:

(101) For fatigue verification see 4.3.7 of this Part 2.

4.2.3.5 Design of members in prestressed concrete

4.2.3.5.1 General

Replacement of Principle P(1) by:

P(101) This section relates to structures with the following types of tendons:

- bonded tendons
- unbonded internal tendons
- external tendons.

Where in specific cases different procedures have to be adopted for unbonded and external tendons, reference should be made to Appendix 105 of this Part 2.

4.3 Ultimate limit states

4.3.1 Ultimate limit states for bending and longitudinal force

4.3.1.1 General

Replacement of Application Rule (5) by:

(105) The actual total width may be taken into account in the design resistance, if adequate connection between flange and web is ensured by shear reinforcement.

4.3.1.3 Brittle failure and hyperstrength

Addition after Application Rule (3):

(104) Where specific measures to avoid brittle failure due to stress-corrosion of prestressing steel are considered necessary, the rules given below should be used.

(105) For prestressed structures, Principle P(1) in ENV 1992-1-1 may be satisfied by any one of the following alternative methods:

a) hypothetically reducing the number of tendons to a number such that the cracking moment, calculated assuming a tensile strength $f_{ctk;0.05}$, is less than or equal to the moment due to the frequent combination of actions, and then ensuring that, with this reduced number of tendons, the ultimate bending resistance exceeds the moment due to the frequent combination of actions.
For this verification the moment redistribution can be considered.
The ultimate bending resistance should be calculated on the basis of the γ_M-factors for the accidental design situation.

b) ensuring that the integrity of the tendons can either be monitored or checked by a proven external inspection technique

c) providing a minimum reinforcement area according to Equation (4.184):

$$\min A_s = \frac{M_{r,ep}}{f_{yk}\, z_s} \qquad (4.184)$$

where:

$M_{r,ep}$ is the cracking moment calculated assuming a tensile stress equal to $f_{ctk0.05}$, according to Table 3.1 in ENV 1992-1-1, at the extreme tension fibre of the section ignoring any action of prestressing. For joints between segmental precast elements, $f_{ctk0.05}$ should be taken equal to zero.

z_s is the lever arm at the ultimate limit state related to the steel reinforcement; for rectangular sections, it may be taken as $z_s = 0{,}9 \cdot d$.

(106) In cases where method c) in (105) above is used, the following rules apply.

(107) The minimum reinforcement area min A_s should be provided in areas where under the infrequent combination of actions tensile stresses in the concrete occur. For this check, the isostatic effects of prestress should be neglected, but the hyperstatic effects should be considered.

(108) For pretensioned members, Equation (4.184) above should be applied as follows:

Either,

- those tendons with a concrete cover that is at least |2| times the minimum concrete cover according to 4.1.3.3 may be taken into account. In this case, f_{yk} should be replaced by $f_{po.1k}$;

or,

- all tendons with f_{yk} taken as not greater than 500 N/mm², may be taken into account.

In both cases, z_s relates to the pretensioned tendons.

(109) For the satisfaction of Principle P(1) in ENV 1992-1-1, the structure should be designed such that adequate ductility is ensured. For this purpose and in the absence of other appropriate measures, the minimum reinforcement area min A_s defined by Equation (4.184) in the spans of continuous T-beams and box sections should extend to the supports of the span considered.

(110) However, in box sections this extension is not necessary if at the ultimate limit state the resisting tensile capacity over the supports provided by the reinforcing steel and the tendons, calculated with the characteristic values f_{yk} and 0,9 f_{pk} respectively, is less than the resisting compressive capacity of the bottom flange, i.e. failure of the compressive zone is not likely to occur:

$$A_s f_{yk} + A_p \times 0{,}9 f_{pk} < t_{inf}\, b_0 \times 0{,}85 f_{ck} \tag{4.185}$$

where:

t_{inf} is the thickness of the bottom flange of the box section

b_0 is the width of the bottom flange of the box section

A_s, A_p denote the area of the reinforcing and prestressing steel respectively, in the tensile zone at the ultimate limit state.

4.3.2 Shear

4.3.2.5 Shear between web and flanges

Replacement of Application Rule (6) by:

(106) In the case of flanges or webs subjected to combined shear and transverse bending with or without in-plane direct stresses, the element should be considered as divided into two layers. The in-plane forces should be divided proportionately between the two layers whilst the bending moment should be transposed into an equivalent couple with its forces acting at the centroids of the layers.
Reinforcement should be provided to ensure equilibrium of each layer and may be calculated using an appropriate strut and tie model. The angle of the concrete struts to the reinforcement is limited to:

$$|0{,}5| \leq \cot\theta \leq |2{,}0|$$

4.3.5 Ultimate limit states induced by structural deformation (buckling)

4.3.5.2 Design procedures

Replacement of Principle P(3) by:

P(103) Possible uncertainties concerning the restraints at connections shall be considered.

4.3.5.3 Classification of structures and structural elements

4.3.5.3.1 General

Replacement of Principle P(1) by:

P(101) For the purpose of design calculations, structures or structural members may be classified as non-sway or sway depending on their sensitivity to second order effect due to displacements perpendicular to the direction of the compressive force.

4.3.5.3.2 Bracing elements and braced structures

Replacement of Application Rules (1) to (3) by:

(101) Application Rules (1) to (3) in 4.3.5.3.2 of ENV 1992-1-1 do not apply to bridges.

4.3.5.3.4 Isolated columns

Replacement of Application Rule (1) by:

(101) These may be either:

- isolated compression members
- compression members which are integral parts of a structure but which are considered to be isolated for design purposes.

4.3.5.3.5 Slenderness of isolated columns

Replacement of Application Rule (1) by:

P(101) For bridges, the effective height or length of a column l_o shall be calculated taking into account the soil-structure interaction and the support conditions at the top and bottom of the column.

4.3.5.4 Imperfections

Addition after Application Rule (3):

(104) When calculation the inclination υ using Equation (2.10) in ENV 1992-1-1, the minimum values specified in 2.5.1.3 (4) should not be applied.

(105) Thermal effects due to temperature differences over the cross section of piers may be considered as initial imperfections, additionally to those specified in ENV 1992-1-1.

4.3.5.5 Specific data for different types of structure

Replacement of subclauses 4.3.5.5.1 to 4.3.5.5.3 by:

P(101) Subclause 4.3.5.5 in ENV 1992-1-1 does not apply to bridges.

Additional clauses after 4.3.5.7:

4.3.6 Verification under impact

4.3.6.1 General

P(101) The following clauses contain provisions concerning impact on structures only. Safety barriers, guard rails, parapet walls etc. shall be designed in accordance with ENV 1991-3, and in agreement with the owner and/or the responsible authority.

P(102) Impact actions shall be considered as accidental actions.

4.3.6.2 Measures

(101) In the absence of national rules, the following ones are applicable.

(102) Priority should be given to preventive measures which avoid or reduce the risk of impact.

(103) The effectiveness of shielding structures placed in front of the load bearing structure should be verified.

(104) Expendable members should lose their ability to transmit considerable horizontal forces after failure under impact.

(105) The deformations of the structure after failure of an expendable structural member should be checked to ensure clearance under the deformed structure.

(106) If the danger of impact cannot be eliminated and if a design with expendable members is not possible, the structural safety should be ensured by designing the structural members to have adequate resistance.

4.3.6.3 Detailing

P(101) In the absence of specific dynamic investigations, the impacted structural members shall be designed such that they can resist also an impact in the opposite direction.

P(102) In all cases, where plastic deformations of the impacted structural members are considered to absorb a substantial proportion of the kinetic energy, the ultimate strains shall be checked. The reinforcement shall not be welded in this case.

(103) The concrete cover outside the outer longitudinal reinforcement should not be taken into account for the determination of the ultimate capacity of the collision zones.
Compression reinforcement can only be considered in the design when restrained against buckling (see 5.4.1.2.2 in ENV 1992-1-1).

(104) Appropriate stirrup reinforcement with small spacing should be provided. For very high impact forces even steel encasement to a collision zone may be required.

Insert new subclause after 4.3.6:

4.3.7 Verification of fatigue

4.3.7.1 Verification conditions

P(101) Structural members which are subjected to a significant stress variation shall be designed for fatigue. In this case, the verification shall be performed separately for concrete and steel.

(102) A fatigue verification is generally not necessary for structures and structural components such as:

a) footbridges;

b) buried arch and frame structures with a minimum earth cover of |1,0 m| for road bridges and |1,5 m| for railway bridges;

c) foundations;

d) piers and columns, which are not rigidly connected to superstructures;

e) retaining walls of road bridges[1];

f) abutments of road bridges[1] which are not rigidly connected to superstructures (except the slabs and walls of hollow abutments);

g) concrete in compression for road bridges if 4.4.1.1 (103) in this Part 2 is satisfied;

h) steel and prestressed reinforcement without welded connections or couplers for bridges designed according to category A, B, and C of Table 4.118 of this Part 2;

i) prestressing and reinforcing steel with welded connections or couplers in those regions where, under the frequent combination of actions with a reduction factor of 0,85 applied to the characteristic value of the prestressing force, P_k, only compression stresses occur in the extreme fibres.

4.3.7.2 Combination of actions and partial safety factors for fatigue verification

(101) The partial safety factors for load and model uncertainties of action effects should be taken as

$$\gamma_F = |1{,}0| \quad \text{and} \quad \gamma_{Sd} = |1{,}0| \tag{4.186}$$

[1] to be checked for railway bridges

(102) Partial safety factors for material properties are given in Table 4.115.

Table 4.115: Partial safety factors for material properties for fatigue verification

Verification for	Concrete $\gamma_{c,fat}$	Steel Reinforcement, Tendons $\gamma_{s,fat}$
safety factor	|1,5|	|1,15|

P(103) In general, fatigue verification for steel and concrete shall be performed taking into account the effects of the following combination of actions:

- permanent actions
- characteristic value of the prestressing force (see 2.5.4.2)
- most unfavourable value of settlements (appropriate estimate values)
- most unfavourable frequent value of temperature
- relevant fatigue traffic load model (see ENV 1991-3 and Appendix 106)
- where relevant, wind fluctuations.

However, when using the methods given in 4.3.7.4 for concrete, the frequent combination of actions should be applied.

In the absence of a more refined verification method in coupling joints the characteristic value of the prestressing force shall be reduced by a factor of 0,85.

4.3.7.3 Internal forces and stresses for fatigue verification

P(101) The stress calculation shall be based on the assumption of cracked cross sections neglecting the tensile strength of concrete but satisfying compatibility of strains (plane sections remain plane).

(102) Internal forces may be calculated using linear elastic models throughout the structural elements considered. In cracked zones a reduced stiffness may be taken into account.

(103) For the calculation of internal forces according to (102) above and stresses (see P(101) above), the modular ratio may be taken as $\alpha = 10$.

P(104) For cracked cross sections subjected to fatigue the effect of the different bond behaviour of prestressing and reinforcing steel shall be taken into account, see 4.3.7.6.

P(105) For structural members with shear reinforcement, the determination of the forces in the reinforcement and in the concrete shall be carried out using the truss model.

(106) When using the standard method for fatigue verification of the shear reinforcement the contribution V_{cd} of the concrete to V_{Rd3} should be taken not greater than 0,5 V_{Rd1}.

(107) When action effects due to shear are determined using the variable strut inclination method, the inclination of the compression struts θ_{fat} may be taken according to Equation (4.187):

$$\tan \theta_{fat} = \sqrt{\tan \theta} \leq 1{,}0 \tag{4.187}$$

where:

θ is the angle of concrete compression struts to the beam axis assumed in the design for shear at the ultimate limit state according to 4.3.2.4 in ENV 1992-1-1.

4.3.7.4 Fatigue verification for concrete under compression, shear and punching shear

(101) For concrete under compression adequate fatigue resistance may be assumed if Equation (4.188), shown graphically in Figure 4.134, is satisfied. Otherwise, a more refined fatigue verification may be necessary (see Appendix 106 for railway bridges).

$$\frac{\sigma_{c,max}}{f_{cd}} \leq 0{,}5 + 0{,}45\,\frac{\sigma_{c,min}}{f_{cd}} \leq 0{,}9 \tag{4.188}$$

where:

$\sigma_{c,max}$ is the maximum compressive stress at a fibre under the frequent combination of actions

$\sigma_{c,min}$ is the minimum compressive stress at the same fibre where $\sigma_{c,max}$ occurs

If $\sigma_{c,min} < 0$ (tension) then $\sigma_{c,max}/f_{cd} \leq 0{,}5$ should be fulfilled.

The increase of the reference compressive strength with the age of the concrete at the time t_0 before the cyclic loading occurs, may be taken into account by applying the factor $\beta_{cc}(t_0)$ to the design concrete strength f_{cd}. For $\beta_{cc}(t_0)$ see 4.4.3.2 (102) in this Part 2.

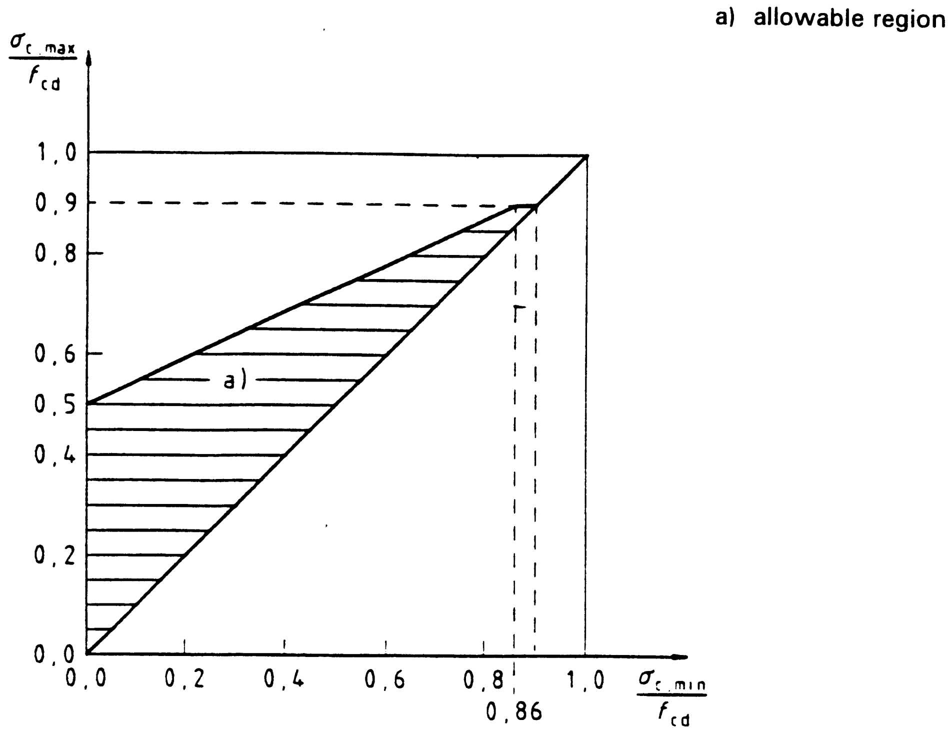


Figure 4.134: Allowable stress variation for concrete under compression according to Equation (4.188) without an explicit fatigue verification

(102) Equation (4.188) and Figure (4.134) apply also to the compression struts of members subjected to shear. In this case the reference strength f_{cd}, should be reduced by the effectiveness factor υ given by Equation (4.21) in ENV 1992-1-1.

(103) In members without shear reinforcement, adequate fatigue resistance of concrete under shear may be assumed if either Equation (4.189) or (4.190), illustrated graphically in Figure 4.135, is satisfied. Otherwise, a more refined fatigue verification may be necessary.

$$\text{for } \frac{\tau_{min}}{\tau_{max}} \geq 0: \quad \left|\frac{\tau_{max}}{\tau_{Rd1}}\right| \leq 0{,}5 + 0{,}45\left|\frac{\tau_{min}}{\tau_{Rd1}}\right| \leq 0{,}9 \qquad (4.189)$$

$$\text{for } \frac{\tau_{min}}{\tau_{max}} < 0: \quad \left|\frac{\tau_{max}}{\tau_{Rd1}}\right| \leq 0{,}5 - \left|\frac{\tau_{min}}{\tau_{Rd1}}\right| \qquad (4.190)$$

where:

τ_{max} is the maximum nominal shear stress under frequent combination of actions.

τ_{min} is the minimum nominal shear stress under frequent combination of actions at the section where τ_{max} occurs.

τ_{Rd1} $= V_{Rd1}/(b_w \cdot d)$ with the design shear resistance V_{Rd1} according to Equation (4.18) in ENV 1992-1-1.

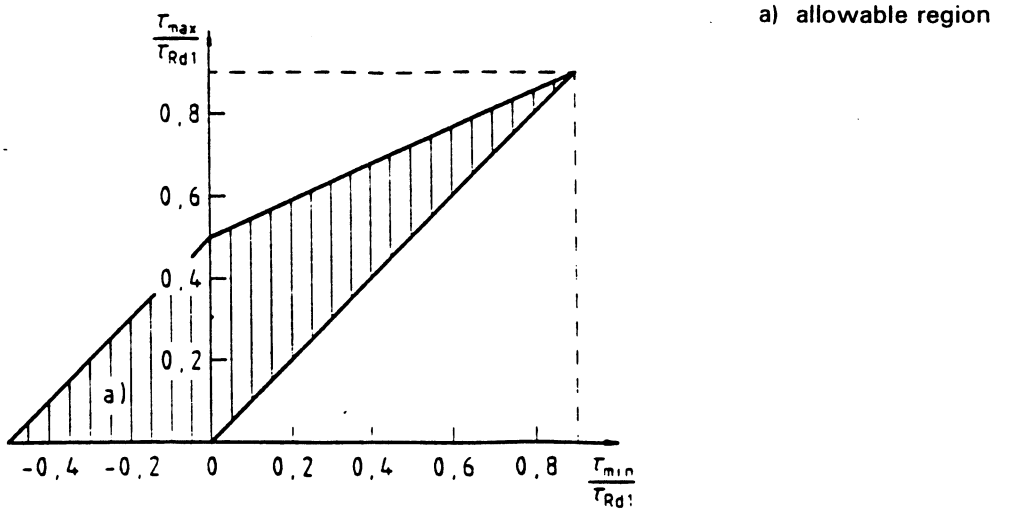


Figure 4.135: Allowable shear stress variation for members without shear reinforcement according to Equations (4.189) and (4.190)

(104) In the case of punching shear, the maximum and minimum design shear stresses should satisfy Equations (4.189) and (4.190) respectively
For the calculation of the design shear stresses clause 4.3.4.2 of ENV 1992-1-1 applies, i.e. $\tau = v_{Sd}/d$ and $\tau_{Rd1} = v_{Rd1}/d$.

4.3.7.5 Fatigue verification for prestressing and reinforcing steel

(101) For unwelded reinforcing bars subjected to tension, adequate fatigue resistance may be assumed if, under the frequent combination of actions, the stress variation, $\Delta\sigma_s$, does not exceed |70| N/mm².

(102) For reinforcing or prestressing steel and couplers adequate fatigue resistance may be assumed if the following expression is satisfied:

$$\gamma_F \ \gamma_{Sd} \ \Delta\sigma_{s,equ} \leq \frac{\Delta\sigma_{Rsk}\,(N^*)}{\gamma_{s,fat}} \tag{4.191}$$

where:

$\Delta\sigma_{Rsk}(N^*)$ is the stress range at N^* cycles from the appropriate S-N lines given in 4.3.7.7 or 4.3.7.8 of this Part 2.

$\Delta\sigma_{s,equ}$ is the damage equivalent stress range defined as the stress range of a constant stress spectrum with N^* stress cycles which results in the same damage as the spectrum of stress ranges caused by flowing traffic loads.

(103) For decks of road and railway bridges the damage equivalent stress range $\Delta\sigma_{s,equ}$ may be calculated by the procedure given in Appendix 106.

(104) For cases not defined in Appendix 106, it should be verified that the fatigue damage factor D_{Sd} of steel caused by the relevant fatigue traffic loads satisfies the condition:

$$D_{Sd} \leq 1 \tag{4.192}$$

For the calculation of the fatigue damage factor D_{Sd}, the Palmgren-Miner-Rule applies. Appropriate S-N lines for steel should be taken from 4.3.7.7 or 4.3.7.8 of this Part 2 reduced by $\gamma_{s,fat}$.

(105) For fatigue verification according to (104) above the Fatigue Load Models 4 and 5 according to ENV 1991-3 may be used for road bridges as relevant fatigue traffic load.

(106) Application Rules (101) to (105) above also apply to shear reinforcement.

4.3.7.6 Effects of different bond behaviour of prestressing and reinforcing steel

P(101) The effect of different bond behaviour of prestressing steel and reinforcing steel shall be taken into account for the stress calculation in the reinforcing steel.

(102) The effect of different bond behaviour of prestressing steel and reinforcing steel may be accounted for by increasing the conventional steel stress in the reinforcing steel by the factor

$$\eta = \frac{A_s + A_p}{A_s + \xi_1 \, A_p} \tag{4.193}$$

where:

ξ_1 is the adjusted ratio of bond strength according to 4.4.2.3 in this Part 2.

(103) Unless the method given in (102) above is used, the conventional steel stress should be calculated according to 4.3.7.3 (102) and (103), but with a reduction factor ξ_1 applied to the modulus of elasticity of prestressing steel.

(104) For the verification in design situations where curved tendons are used, ξ_1 should be taken as 1,0.

4.3.7.7 Fatigue strength of prestressing steel

(101) The parameters defining the fatigue strength of prestressing steel should be taken from Table 4.116 (see also Figure 4.136).

Table 4.116: Parameters for the S-N curves of prestressing steel

S-N curve of prestressing steel used for		stress exponent		$\Delta\sigma_{Rsk}$ [N/mm²] at N =	
	N^*	k_1	k_2	N^*	2 x10⁶
pretensioning:	10^6	5	9	185	170
post-tensioning:					
- single strands in plastic ducts	10^6	5	9	185	170
- curved tendons in plastic ducts and straight tendons	10^6	5	9	160	145
- curved tendons in steel ducts	10^6	3	7	120	110
- couplers [a])	10^6	3	5	80	70
[a]) unless other S-N-curves can be justified by test results					

(102) Verification of unbonded internal or external tendons for fatigue is not necessary.

(103) The S-N curves according to Figure 4.136 generally follow the Equation $(\Delta\sigma_{Rsk})^m N$ = constant for both $N < N^*$ and $N > N^*$ with corresponding exponent $m = k_1$ and $m = k_2$.

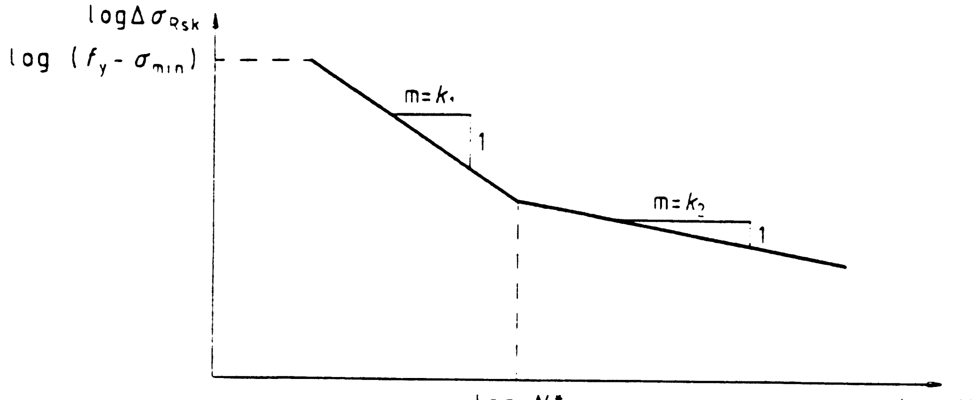


Figure 4.136: Shape of the characteristic fatigue strength curves (S-N curves) for prestressing steel

4.3.7.8 Fatigue strength of reinforcing steel

(101) The parameters defining the fatigue strength of reinforcing steel should be taken from Table 4.117.

Table 4.117: Parameters of S-N curves for reinforcing steel

Type of reinforcement		stress exponent		$\Delta\sigma_{Rsk}$ [N/mm²] at N =	
	N^*	k_1	k_2	N^*	2 x 10^6
Straight and bent bars for $D \geq 25\ \varnothing$ for $D < 25\ \varnothing$ see [a])	10^6	5	9	195	180
Welded bars including tack welding and butt joints[b]) couplers [b])	10^7	3	5	60	100

$\varnothing$ = bar diameter
D = diameter of mandrel

[a]) Values of $\Delta\sigma_{Rsk}$ are those of the appropriate straight bar. Values for a bar of diameter $\varnothing$ with a bend diameter $D < 25\ \varnothing$ should be obtained by multiplying the straight bar values by a reduction factor $\zeta = 0{,}35 + 0{,}026\ D/\varnothing$.
For stirrups ζ should be taken as 0,9.

[b]) Unless other S-N curves can be justified by test results.

(102) Special attention should be paid to fatigue effects in a corrosive environment as defined ENV 1992-1-1, Table 4.1, Exposure classes 4 and 5.

4.4 Serviceability limit states

4.4.0 General

4.4.0.2 Scope

Replacement of this subclause by:

P(101) This chapter covers the common serviceability limit states.
These are:

- stress limitation (see 4.4.1)
- crack and decompression control (see 4.4.2)
- deflection control (see 4.4.3)
- vibration control (see 4.4.4).

Other limit states (such as tightness against fluid penetration) may be of importance in particular structures but these are not covered in this code.

Addition after subclause 4.4.0.2:

4.4.0.3 Classification of verification criteria

P(101) The bridge or specific parts of it shall be classified into environmental classes according to Table 4.1 in ENV 1992-1-1.

(102) In order to ensure the performance required, the verification criteria for the bridge or for specific parts of it, may be classified into verification categories according to Table 4.118. The appropriate category should be agreed with the client. Table 4.118 applies to both the construction phases and persistent situations.

(103) Table 4.118 contains the criteria for the verification of decompression and crack width. When using this Table, both criteria need to be verified.

Table 4.118: Classification of verification criteria

category	Combination of actions for the verification of Decompression	Crack width
A	infrequent	-
B	frequent	infrequent
C	quasi-permanent	frequent
D	-	frequent
E	-	quasi-permanent

4.4.1 Limitation of stresses under serviceability conditions

Replacement of subclause 4.4.1.1 in ENV 1992-1-1 by:

4.4.1.1 Basic considerations

P(101) Excessive creep and microcracking shall be avoided by limiting the compressive stress under service load.

(102) For prestressed concrete the maximum compressive stress at transfer should be limited to |0,6| $f_c(t)$. When the compressive stress of concrete exceeds |0,45| $f_c(t)$, nonlinear creep has to be considered. $f_c(t)$ denotes the mean value of the concrete compressive strength at the time t at transfer.

(103) The compressive stress in the concrete due to the infrequent combination of actions and the characteristic value of prestress should be limited to |0,6| f_{ck}. This limit can be relaxed if the compression zone is confined by, for example, transverse reinforcement in excess of 1% of the volume of the compression zone.

For precast elements which are subject to an adequate control system (see, for example, chapter 7 in ENV 1992-1-1), the value |0,6| $f_c(t)$ may be exceeded during construction by |10%| if strict control of strength and separate control of loss of prestress are provided.

P(104) The steel stress shall be such that inelastic strains in the steel are avoided.

(105) The tensile stress in the steel reinforcement under the infrequent combination of actions should not exceed |0,8| f_{yk}.

(106) The stress in prestressing tendons under the quasi-permanent combination of actions should not exceed |0,65| f_{pk} after allowance for all losses.

4.4.1.2 Methods for checking stresses

Replacement of Application Rules (2) and (3) by:

(102) In general, the stress limitations in 4.4.1.1 should be checked.

(103) Long term effects may be taken into account by assuming a modular ratio in the range of 10 to 15 according to the ratio of permanent to variable actions.

Addition after Application Rule (8):

(109) For the check of crack width and stresses, the effective width given in 2.5.2.2 in this Part 2 should be applied.

4.4.2 Limit states of decompression and of cracking

4.4.2.1 General considerations

Replacement of Application Rules (6) to (8) and Principle P(9) by:

(106) When designing to categories A, B and C of Table 4.118 for the limit state of decompression no tensile stresses in the concrete are allowed under the relevant combination of actions.

(107) For crack width control, the deemed to satisfy criteria given in Tables 4.120 and 4.121 below are based on the following design crack widths:

0,2 mm for prestressed concrete
0,3 mm for reinforced concrete.

(108) In design situations where the design crack width differs from the values given in (107), the maximum bar diameter to control the crack width can be determined with Equation (4.200) in 4.4.2.3 (106).

(109) For design crack width, externally prestressed members without internal tendons may be treated as ordinary reinforced concrete members.

Replacement of subclauses 4.4.2.2, 4.4.2.3 and 4.4.2.4 in ENV 1992-1-1 by:

4.4.2.2 Minimum reinforcement areas

4.4.2.2.1 General

P(101) For the reasons of durability and appearance of the concrete a minimum reinforcement area shall be provided in reinforced bridge structures in order to prevent wide single cracks due to imposed deformations not considered in the design calculation, self-equilibrating stresses, or distorsion of prestress.

(102) P(101) above may be considered satisfied, where a minimum reinforcement area is provided in the outer parts of the cross section. This minimum reinforcement area may be taken into account for all verifications for the ultimate and serviceability limit states.

(103) The maximum bar spacing of the minimum reinforcement should not exceed |200 mm|.

4.4.2.2.2 Regions of minimum reinforcement

(101) The minimum reinforcement area according to Equation (4.194) should be placed in areas where, under the infrequent combination of actions, the concrete stresses are tensile or less than 1 N/mm² compressive.

4.4.2.2.3 Minimum reinforcement areas

(101) The minimum reinforcement ratio may be calculated from the relation given below. For box girders and T-beams the minimum reinforcement ratio should be calculated separately for webs and flanges.

$$\rho_s + \xi_1 \ \rho_p = \frac{0{,}8 \ k_c \ k \ f_{ctm}}{\sigma_s} \qquad (4.194)$$

where:

ρ_s ratio of the area of reinforcing steel to the area of the tensile zone of the part of the cross section under consideration: $\rho_s = A_s/A_{ct}$.

ρ_p ratio of the area of prestressing steel within an area of not more than 300 mm around the steel reinforcement in the tensile zone to the area of the tensile zone of the part of the cross section under consideration: $\rho_p = A_p/A_{ct}$.

f_{ctm} concrete tensile strength; for joints between segmental precast elements f_{ctm} should be taken equal to zero.

A_{ct} area of the tensile zone immediately prior to cracking of the cross section, web or flange as appropriate.

σ_s steel stress in the minimum reinforcement area according to Table 4.120. σ_s may be increased by a factor $\eta = \sqrt{f_{ctm} / f^*_{ctm}}$

where $f^*_{ctm} = 2{,}5$ N/mm² and f_{ctm} is the actual tensile strength of the concrete. σ_s should not exceed kf_{yk}.

ξ_1 adjusted ratio of bond strength according to 4.4.2.3.

k a coefficient which takes account of the effect of secondary crack formation which leads to a reduction of restraint forces:

$k = 1{,}0$ for webs with $h \leq 0{,}3$ m or flanges with widths less than 0,3 m
$k = 0{,}65$ for webs with $h \geq 0{,}8$ m or flanges with widths greater than 0,8 m

intermediate values may be interpolated.

k_c a coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking and of the lever arm in the cracked state.

- for pure tension

$k_c = 1{,}0$

- for rectangular sections and webs of box sections and T-beams

$$k_c = 0{,}4 \left(1 + \frac{N_{Sd}}{k_1 \; b \; h \; (h / h^*) f_{ctm}} \right) \quad (4.195)$$

with

$h^* = h$ for $h < 1{,}0$ m
$h^* = 1{,}0$ m for $h \geq 1{,}0$ m

- for flanges of box sections and T-beams

$$k_c = 0{,}9 \; \frac{F_{cr}}{A_{ct} \; f_{ctm}} \geq 0{,}5 \quad (4.196)$$

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

k_1 a coefficient considering the effects of axial forces to the stress distribution.

$k_1 = 1{,}5$ where N_{Sd} is a compressive force

$k_1 = \frac{2 \, h^*}{3 \, h}$ where N_{Sd} is a tensile force

F_{cr} tensile force within the flange immediately prior to cracking due to the cracking moment calculated with f_{ctm}.

(102) For slabs with variable thickness the minimum reinforcement related to the mean thickness can be distributed uniformly. For tension chords of T-beams or box sections the minimum reinforcement should refer to the relevant thickness.

4.4.2.3 Control of cracking

(101) The minimum reinforcement according to Equation (4.194) is adequate to control the crack width according to 4.4.2.1 (107) if it can be verified that under the relevant combination of actions for controlling the crack width according to Table 4.118 the concrete stress in the extreme fibre of the section does not exceed the mean value of the concrete tensile strength ($\sigma_c \leq f_{ctm}$).

(102) If the concrete stress in the extreme fibre of the section under the relevant combination of actions for controlling the crack width according to Table 4.118 does exceed the mean value of the concrete tensile strength ($\sigma_c > f_{ctm}$) one of the following methods below for controlling the crack width should be applied.

(103) In design cases according to (102) above, the crack width may be considered adequately controlled if either the bar diameter does not exceed the values given in Table 4.120 or the maximum bar spacing does not exceed the limit in Table 4.121.

In these Tables, σ_s is the stress in the reinforcing steel, unless there is prestressing steel alone, in which case σ_s is equal to $\Delta\sigma_p$.

The steel stress for the application of Tables 4.120 and 4.121 should be calculated under the relevant combination of actions using Equations (4.198) and (4.199) respectively.

(104) In the case of rectangular sections or webs of T-beams and box girders the maximum bar diameter ϕ_s^* given in Table 4.120 may be modified as follows:

$$\phi_s = \phi_s^* \; \frac{h_{cr}}{10 \; (h - d)} \geq \phi_s^* \quad (4.197)$$

where:

ϕ_s is the adjusted maximum bar diameter

ϕ_s^* is the maximum bar diameter in Table 4.120

h_{cr} depth of the tensile zone immediately prior to cracking considering characteristic value of prestress and axial forces under the quasi-permanent combination of actions.

h is the overall depth of the section

d is the effective depth to the centroid of the outer layer of reinforcement.

(105) For the control of crack width the steel stress may be calculated taking into account the different bond behaviour of prestressing and reinforcing steel:

$$\sigma_s = \sigma_s^{II} + 0{,}4\ f_{ctm} \left(\frac{1}{eff\,\rho_p} - \frac{1}{eff\,\rho_{tot}} \right) \tag{4.198}$$

$$\Delta\sigma_p = \sigma_s^{II} - 0{,}4\ f_{ctm} \left(\frac{1}{eff\,\rho_{tot}} - \frac{\xi_1^2}{eff\,\rho_p} \right) \tag{4.199}$$

(106) The maximum bar diameter to control the crack width w_k may be calculated from:

$$\phi_s = \frac{3{,}6\ w_k\ eff\rho_p\ E_s}{\sigma_s - 0{,}4\ \dfrac{f_{ctm}}{eff\rho_p}\ (1 + \alpha_e\ eff\rho_p)} \tag{4.200}$$

with the following notation in Equations (4.198) to (4.200):

$$eff\rho_p = \frac{A_s + \xi_1^2\ A_p}{A_{c,eff}} \tag{4.201}$$

$$eff\rho_{tot} = \frac{A_s + A_p}{A_{c,eff}} \tag{4.202}$$

α_e Modular ratio ($\alpha_e = E_s/E_{cm}$)

σ_s^{II} steel stress in the reinforcing steel or prestressing steel, respectively, calculated in the cracked state assuming full bond under the relevant combination of actions.

$A_{c,eff}$ effective area of concrete which is relevant for crack width control

$$A_{c,eff} = 2{,}5\ b\ (h - d) \leq \frac{F_{cr}}{f_{ctm}} \tag{4.203}$$

b width of the section in the region of A_s and A_p.

F_{cr} tensile force within the tensile zone immediately prior to cracking due to the cracking moment calculated with f_{ctm} considering characteristic values of prestress and axial forces under the quasi-permanent combination of actions.

A_s area of reinforcement within the effective area $A_{c,eff}$

A_p area of pre- or post-tensioned tendons within the effective area $A_{c,eff}$

ξ_1 adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel and may be calculated by:

$$\xi_1 = \sqrt{\xi \frac{\phi_s}{\phi_p}} \tag{4.204}$$

If only prestressing steel is used to control the crack width then $\xi_1 = 1{,}0$.

ϕ_s largest diameter of steel reinforcement.

ϕ_p equivalent diameter of prestressing steel;

$\phi_p = 1{,}6 \cdot \sqrt{A_p}$ for tendons with several strands or wires

$\phi_p = 1{,}75\ \phi_{wire}$ for single strands with 7 wires

$\phi_p = 1{,}20\ \phi_{wire}$ for single strands with 3 wires

ξ ratio of bond strength of prestressing steel and high bond reinforcing steel. In the absence of appropriate data ξ may be taken from Table 4.119.

Table 4.119: Nominal ratio ξ of mean bond stress of prestressing steel and high bond reinforcing steel for crack control

Type of tendon	Pre-tensioned members	Post-tensioned members
smooth prestressing steel	-	0,4
7-wire strands	0,6	0,5
ribbed prestressing wires	0,8	0,7
ribbed prestressing bars	1,0	0,8

Table 4.120: Maximum diameters ϕ_s^* for high bond bars

Steel stress	Maximum bar diameter ϕ_s^*(mm)	
σ_s (N/mm²)	reinforced sections	prestressed sections
120	--	40
140	40	32
160	32	25
200	25	16
240	20	12
280	16	8
320	12	6
360	10	5
400	8	4
450	6	

If only prestressing steel is used to control the crack width then the ϕ_s^* -values given in Table 4.120 should be multiplied by ξ taken from Table 4.119.

Table 4.121: Maximum bar spacing for high bond bars

	Maximum bar spacing (mm)		
Steel stress σ_s (N/mm²)	Pure flexure (reinforced sections)	Pure tension (reinforced sections)	Prestressed sections (bending)
160	300	200	200
200	250	150	150
240	200	125	100
280	150	75	50
320	100	--	--
360	50	--	--

If only prestressing steel is used to control the crack width then the values of bar spacing given in Table 4.121 should be multiplied by 0,5.

4.4.3 Limit states of deformation

Replacement of this subclause in ENV 1992-1-1 by:

4.4.3.1 Basic considerations

P(101) The deformation of a structure or a member of a structure shall not be such that it adversely affects its proper function or appearance.

P(102) For railway bridges, clause G3 in ENV 1991-3 is applicable. For other bridges, limiting values of deformations in the presence of traffic should, where relevant, be agreed with the client, together with the associated combinations of actions.

P(103) Deformations shall not exceed those which can be accommodated by connections, joints and bearings.

P(104) The deformations during construction shall be controlled such that:

the concrete is not impaired during its placing and setting

a regular longitudinal profile is achieved,

the required long-term geometry is achieved.

(105) In order to avoid cracking in the concrete during concreting, the deflection of the scaffolding or centering should be limited to

$$(L + 40) / 2000 \tag{4.205}$$

where: L is the effective span in metres.

This limit may be increased to $L/300$, if cracking during concreting in controlled.

(106) If deformations of a structure are determined by calculation, they should be calculated considering the quasi-permanent combination of actions. Material properties should be taken as mean values. The effects of creep, shrinkage and cracking should be taken into account.

(107) In the absence of other information, the calculation of deformations during construction should be based on the characteristic combination of actions associated with the mean value of prestress.

4.4.3.2 Checking deflections by calculation

P(101) Deflections shall be calculated using an elastic modulus appropriate to the actual concrete in the structure.

(102) The creep effects may be calculated using the visco-linear model according to 2.5.5.1 (5) and using Equation (2.21) in ENV 1992-1-1 with $E_c(t_0)$ defined as

$$E_c\,(t_0) = \beta_E\,(t_0)\;\;E_{c28} \tag{4.206}$$

where:

$$\beta_E\,(t_0) = \sqrt{\beta_{cc}(t_0)} \tag{4.207}$$

$$\beta_{cc}(t_0) = \exp\left\{s\left[1 - \sqrt{\frac{28}{t_0 / t_1}}\right]\right\} \tag{4.208}$$

s = 0,20 for rapid hardening high strength cements
= 0,25 for rapid hardening normal strength cements
= 0,38 for slowly hardening cements

t_1 reference time: t_1 = 1 day

t_0 is the time at initial loading of the concrete (in days)

P(103) The calculation method adopted shall represent the real behaviour of the structure under the relevant actions to an accuracy appropriate to the objectives of the calculation.

(104) Structures designed to categories A, B or C according to Table 4.118 may be considered as uncracked when calculating their deflections. Other structures may be assumed to be cracked.

4.4.3.3 Other components of displacements

(101) Rotations and horizontal displacements should be determined assuming the mean values of the material properties, considering the cracked state, where relevant.
For the relevant combinations of actions and safety elements see EN 1337-1 'Structural bearings'.

New subclause after 4.4.3.3

4.4.4 Limit state of vibration

4.4.4.1 General considerations

P(101) Under the dynamic effects of road, railway, pedestrian, cycle track and wind loading a bridge shall satisfy serviceability limit state criteria including consideration of discomfort to bridge users.

(102) The dynamic effects of wind are not covered by this subclause but should be considered for structures such as cable stayed bridges.

(103) In addition to the dynamic effects of traffic and wind on a complete bridge, the local effects on slender components, such as wide side cantilevers, should be considered.

4.4.4.2 Road bridges

(101) The dynamic effects of standard traffic loads on common types of road bridges for both the ultimate and serviceability limit states may be deemed to be covered by the dynamic factor, which is already included in the characteristic traffic loads.

4.4.4.3 Railway bridges

(101) The dynamic effects of standard railway loading are given in ENV 1991-3.

4.4.4.4 Foot and cycle track bridges

4.4.4.4.1 Design criterion

P(101) The design criterion for bridges with pedestrian and cyclic traffic shall be the avoidance of possible discomfort to the user.

(102) It may be assumed that discomfort to the user will not occur if the maximum vertical acceleration of any part of the deck does not exceed $0{,}5\sqrt{f_0}$ [m/s²], where f_0 is the fundamental natural frequency of the bridge including superimposed permanent load but excluding pedestrian loading.

A fundamental natural frequency in the range 1,6 to 2,4 Hz and, if specified, for the higher range 2,5 to 4,5 Hz should be avoided. If the fundamental natural frequency, f_0, exceeds 5 Hz it may be assumed that the limit state of vibration is automatically satisfied.

4.4.4.4.2 Fundamental natural frequency

(101) The fundamental natural frequency f_0 should be calculated using an uncracked section and the short-term dynamic elastic modulus of the concrete.

(102) Where appropriate, the stiffness of the parapets should be considered where they contribute to the overall flexural stiffness of the deck.

4.4.4.4.3 Acceleration

(101) The maximum vertical acceleration should be calculated assuming that the dynamic loading applied by the pedestrian traffic can be represented by a pulsating point load F, moving across the main span of the deck at a constant speed v as follows:

$$F = 180 \sin (2\pi f_0 T) \quad \text{in Newton} \tag{4.209}$$

$$v = 0{,}9\, f_0 \tag{4.210}$$

where T is the time in seconds and v is the velocity in metre per second

(102) For values of f_0 greater than 4 Hz the calculated maximum acceleration may be reduced by an amount varying linearly from zero reduction at 4 Hz to 70% reduction at 5 Hz.

5 Detailing provisions

5.1 General

Replacement of Application rule (3) by:

(103) This subclause applies also to structures subjected to fatigue loading.

5.3 Prestressing units

5.3.4 Anchorages and couplers for prestressing tendons

Replacement of Application Rule (5) by:

(105) The placing of couplers of more than |50%| of the tendons at one cross section should be avoided, unless

- a continuous minimum reinforcement according to 4.4.2.2.3 is provided, or
- there is a minimum residual compressive stress of |3 N/mm²| under the frequent combination of actions to resist local tensile stresses.

Tendons which are not coupled at one section may not be coupled within a distance less than the value a in Table 5.106 from the relevant cross section.

Table 5.106: Distance of couplers

construction depth h	Distance a
≤ 2,0 m > 2,0 m	1,5 m 3,0 m

P(106) If slabs or carriageway slabs are transversely prestressed, a uniform compressive stress field shall be provided.

(107) A uniform compressive stress field can be provided if each second anchorage device is placed at the edge of the slab.

(108) In an aggressive environment openings and pockets which are necessary to apply the prestress to the tendons should be avoided, where possible, on the upper side of the carriageway slabs.

5.4 Structural members

5.4.2 Beams

5.4.2.1 Longitudinal reinforcement

5.4.2.1.1 Minimum and maximum reinforcement percentage

Replacement of Application Rule (1) by:

(101) The effective cross-sectional area of the longitudinal tensile reinforcement should not be less than that required to control cracking (see 4.4.2.2.3 of ENV 1992-2) and, where necessary, to prevent brittle failure (see 4.3.1.3 of ENV 1992-2).

5.4.2.2 Shear reinforcement

Replacement of Application Rule (7) by:

(107) The maximum longitudinal spacing s_{max} of successive series of stirrups of shear assemblies is defined by the following conditions (with V_{Sd}, V_{Rd1} and V_{Rd2} as defined in 4.3.2 of ENV 1992-1-1):

$$\text{if} \quad V_{Sd} \leq {}^{1}/_{5}\, V_{Rd2} \quad : s_{max} = |0{,}8|\, d \tag{5.117}$$

$$\text{if} \quad {}^{1}/_{5}\, V_{Rd2} < V_{Sd} \leq {}^{2}/_{3}\, V_{Rd2} \quad : s_{max} = |0{,}6|\, d \tag{5.118}$$

$$\text{if} \quad V_{Sd} > {}^{2}/_{3}\, V_{Rd2} \quad : s_{max} = |0{,}3|\, d \tag{5.119}$$

(for V_{Rd2}, see Section 4.3.2.4 Equations (4.25) and (4.26) in ENV 1992-1-1)

Additional subclause after 5.4.8.3:

5.4.9 Precast members

5.4.9.1 General considerations

P(101) Precast concrete members shall be designed and detailed in accordance with ENV 1992-1-3 considering, however, the modifications in this subclause or in relevant product standards.

P(102) The following clauses of ENV 1992-1-3 are specific for buildings and do not apply for bridges:

1.1.2 P(106)	2.1 (106)	2.5.2.1 (107)
2.5.3.5.8	3.5.3	4.1.3.3
4.3.5.6.1 (101)	4.5.3.2 (106)	5.4.7.6
5.4.2.2 (111), (112), (113)	5.4.7.7	5.5

P(103) In clause 4.4.1.1 of ENV 1992-1-3 the infrequent combination of actions shall be used instead of the characteristic combination. In clause 5.4.2.1.1 of ENV 1992-1-3 the reference to ENV 1991-1-1 shall be replaced by ENV 1992-2.

5.4.9.2 Construction joints

5.4.9.2.1 Ultimate limit state of shear

(101) Joints between precast elements, including segmental construction, should be designed in accordance with 4.5.3 of ENV 1992-1-3.
However , 4.5.3.2 (106) does not apply.

5.4.9.2.2 Serviceability limit states

P(101) No decompression of unreinforced contact joints shall be permitted under infrequent combination of actions, including during the erection phase.

(102) For cement mortar joints under frequent combination of actions the stresses in the joint should be compressive throughout the joint and not less than |1,5| N/mm².

(103) In the erection phase, during polymerisation of the mortar, the pressure applied to a resin mortar joint should not be less than |0,15| N/mm² anywhere in the cross section under the characteristic combination of actions; the average pressure should not be less than |0,25| N/mm².

5.4.9.3 Composite members

5.4.9.3.1 General considerations

(101) Clause 5.4.9.3 applies to flexural members consisting of precast concrete units acting with in situ concrete, where provision has been made for the transfer of shear in the contact joint. The precast units may be of either reinforced or prestressed concrete.

P(102) Particular attention shall be given to the effect on stresses and deformation of the method of construction. The structure shall be verified at each stage of construction.

(103) Differential shrinkage between the in situ concrete and precast concrete members may require consideration in analysing composite members for the serviceability limit states (see 5.4.9.3.3), but it need not be considered for the ultimate limit state.

5.4.9.3.2 Ultimate limit states

P(101) Composite members shall be designed for vertical and longitudinal shear in accordance with 4.5.3.2 of ENV 1992-1-3.

5.4.9.3.3 Serviceability limit states

P(101) The stiffness of a composite member shall be determined taking account of the different elastic properties of the precast and in situ concretes.

(102) The effects of differential shrinkage and creep should be taken into account when checking the design criteria of 4.4. Differential shrinkage can be determined from 2.5.5, 3.1.2.5.5, A.1 and A.2 in ENV 1992-1-1.

(103) For composite members comprising prestressed precast units and in situ concrete, which restrains the compression zone of the precast unit, the maximum concrete compressive stress in the precast unit may be increased above the values given in 4.4.1.1 up to |0,75| f_{ck}.

5.5 Limitation of damage due to accidental actions

This clause in ENV 1992-1-1 does not apply to bridges.

6 Construction and workmanship

This clause applies also to ENV 1992-2.

7 Quality control

This clause applies also to ENV 1992-2.

NORMATIVE APPENDICES

APPENDIX 1 (normative)

Additional provisions for the determination of the effects of time-dependent deformation of concrete

Appendix 1 applies also to ENV 1992-2. However, for lightweight aggregate concrete, see ENV 1992-1-4.

APPENDIX 2 (normative)

Non-linear analysis

Appendix 2 applies to ENV 1992-2, provided that adequate ductility of the structure including joints can be ensured.

APPENDIX 3 (normative)

Supplementary information on the ultimate limit states induced by structural deformations

Appendix 3 applies also to ENV 1992-2.

APPENDIX 4 (normative)

Checking deflections by calculation

Appendix 4 applies also to ENV 1992-2.

APPENDIX 105 (normative)

Prestressing with unbonded and external tendons

A105.1 Scope

P(101) This Appendix complements and clarifies the use of **"ENV 1992-1-5 The use of Unbonded and External Prestressing Tendons** in concrete bridges. Unless otherwise stated all references in this Appendix relate to ENV 1992-1-5.

A105.2 Basis of design

A105.2.5 Analysis

A105.2.5.3 Calculation methods

A105.2.5.3.1 Basic considerations

Replacement of Application Rule (108) in ENV 1992-1-5 by:

(108) For internal tendons of short and medium span bridges, with decks consisting of beams or box sections of constant depth, it may be assumed that:

the force in a tendon is constant along a span,

the distributed radial forces are vertical for tendons deviated in the vertical plane.

A105.4 Section and member design

Insert a new subclause:

A105.4.1 Initial prestressing force

P(101) For cables whose profile is contained within the overall depth of the cross section of the prestressed member the initial prestressing force defined in 4.2.3.5.4 P(2) of ENV 1992-1-1 should be applied.

P(102) For external cables whose profile exceeds the overall depth of the cross section of the prestressed member and whose stress range under the frequent combination of actions does not exceed |50| N/mm², the initial prestressing stress shall be less than |0,60| f_{pk}.
Otherwise the external cables shall be considered as stay cables.

A105.4.3 Ultimate limit states

A105.4.3.1 General

A105.4.3.1.5 Internal unbonded tendons

(101) Application Rule (101) in ENV 1992-1-5 does not apply to bridges.

A105.4.3.2 Shear

A105.4.3.2.6 Segmental Construction

Replacement of Application Rules (103) and (104) in ENV 1992-1-5 by:

(103) Under consideration of the infrequent combination of actions and in the absence of justification by former experience, unreinforced joint sections should be compressed over at least |2/3| of their total depth. Special attention should be given to torsion effects in the design of box sections.

A105.4.4 Serviceability limit states

A105.4.4.0 General

A105.4.4.0.3 Load cases and combinations

Replacement of Application Rules (101) to (103) in ENV 1992-1-5 by:

P(101) The classification of the design criteria shall be agreed with the client.

A105.5 Detailing provisions

A105.5.3 Prestressing units

This clause in ENV 1992-1-5 is applicable except as follows:

A105.5.3.1 Arrangement of the prestressing units

Replacement of Principle P(102) in ENV 1992-1-5 by:

P(102) Replacement of the external tendons shall be provided for, if specified by the client.

Addition after Application Rule (110) in ENV 1992-1-5 :

P(111) Accessibility for inspection, monitoring and, where relevant, replacement of the external tendons shall always be provided.

(112) Concerning anchorage systems consideration should be given to anchorage efficiency, fatigue strength, load transfer capacity and corrosion protection and the ability to accommodate angular deviations up to |0,05| rad without noticeable reduction of anchorage efficiency.

A105.5.3.2 Concrete cover

Replacement of Principle P(101) and Application Rule (102) in ENV 1992-1-5 by:

P(101) The concrete cover to internal unbonded tendons shall be the same as specified by 4.1.3.3 in ENV 1992-1-1 for bonded tendons.

A105.5.5 Limitation of damage due to accidental action

A105.5.5.2 Proportioning of ties

Replacement of Principle P(106) in ENV 1992-1-5 by:

P(106) It shall be checked that in the event of a local failure of one tendon the bridge will not collapse under the accidental combination of actions, using the associated γ values.

APPENDIX 106 (normative)

Damage equivalent stresses for fatigue verification

A106.1 General

P(101) This Appendix gives a simplified procedure to calculate the damage equivalent stresses for fatigue verification of superstructures of road and railway concrete bridges on the basis of fatigue load models given in ENV 1991-3. Any load model given in ENV 1991-3 may be used provided that appropriate data for λ values are available.

A106.2 Road bridges

P(101) The values given in this subclause are applicable only to the factored fatigue load model 3 according to ENV 1991-3.
For calculation of the damage equivalent stress ranges for steel verification, the axle loads of the fatigue load model 3 shall be multiplied by factors:

1,75 for verification at the intermediate supports

1,40 for verification in other areas.

P(102) The damage equivalent stress range for steel verification shall be calculated according to:

$$\Delta\sigma_{s,equ} = \Delta\sigma_{s,EC} \quad \lambda_s \tag{A.106.1}$$

where:

$\Delta\sigma_{s,EC}$ stress range caused by fatigue load model 3 (according to ENV 1991-3) with increased axle load as given in P(101), assuming the load combination given in 4.3.7.2.P(103) of this Part 2.

λ_s correction factor to calculate the damage equivalent stress range from the stress range caused by $\Delta\sigma_{s,EC}$

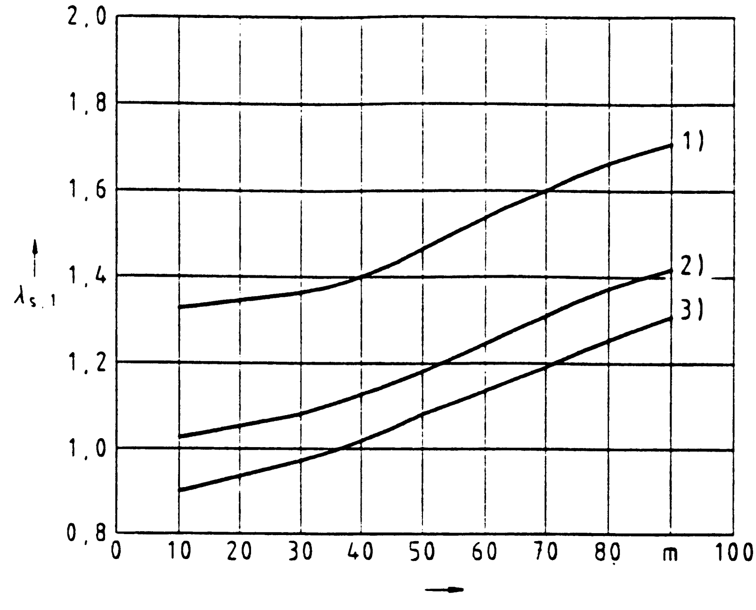
(103) The correction factors λ_s include the influences of span, annual traffic volume, service life, multiple lanes, traffic type and surface roughness and may be calculated by

$$\lambda_s = \varphi_{fat} \quad \lambda_{s,1} \quad \lambda_{s,2} \quad \lambda_{s,3} \quad \lambda_{s,4} \tag{A106.2}$$

where:

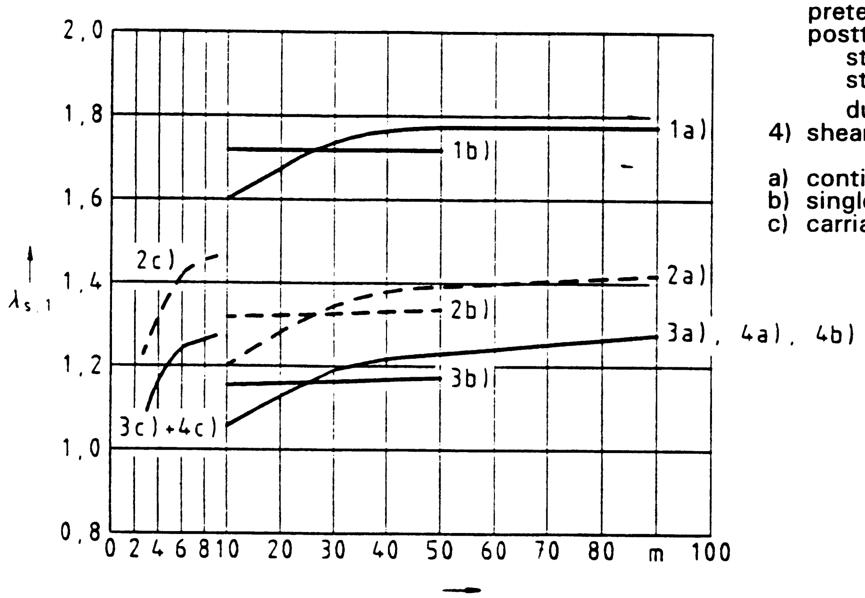
$\lambda_{s,1}$ value for span
$\lambda_{s,2}$ value for annual traffic volume
$\lambda_{s,3}$ value for service life
$\lambda_{s,4}$ value for multiple lanes
φ_{fat} damage equivalent impact factor controlled by the surface roughness as given below.

(104) The $\lambda_{s,1}$ value given in Figures A106.1 and A106.2 denotes the influence of the span L and the shape of S-N curve.



1) splicing devices
2) curved tendons in steel ducts
3) reinforcing steel
   pretensioning (all)
   posttensioning:
   strands in plastic ducts
   straight tendons in steel ducts

Figure A106.1: $\lambda_{s,1}$ value for fatigue verification in the intermediate support area



1) splicing devices
2) curved tendons in steel ducts
3) reinforcing steel
   pretensioning (all)
   posttensioning:
   strands in plastic ducts
   straight tendons in steel ducts
4) shear reinforcement

a) continous beam *)
b) single span beam *)
c) carriageway slab

*) for span $\ell < 10$ m values for $\ell = 10$ m apply

Figure A106.2: $\lambda_{s,1}$ value for fatigue verification in span and for local elements

(105) The $\lambda_{s,2}$ value denotes the influence of the annual traffic volume and traffic type. It may be calculated by Equation (A106.3).

$$\lambda_{s,2} = \overline{Q} \sqrt[k_2]{\frac{N_{obs}}{2.0}} \qquad \text{(A106.3)}$$

where:

N_{obs} number of lorries per year according to ENV 1991-3, Table 4.5 in million.

k_2 slope of the appropriate S-N line to be taken from Tables 4.116 or 4.117 of this Part 2.

Q factor for traffic type according to Table A106.1

Table A106.1: Factors for traffic type α

	Traffic type (see ENV 1991-3, Table 4.7)		
Q factor for	long distance	medium distance	local traffic
$k_2 = 5$	1,0	0,90	0,73
$k_2 = 7$	1,0	0,92	0,78
$k_2 = 9$	1,0	0,94	0,82

(106) The $\lambda_{s,3}$ value denotes the influence of the design working life and may be calculated from Equation (A106.4).

$$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{Years}}{100}} \qquad \text{(A106.4)}$$

where:

N_{Years} design working life of the bridge (to be specified, if different from 100 years).

(107) The $\lambda_{s,4}$ value denotes the influence for multiple lanes and may be calculated from Equation (A106.5).

$$\lambda_{s,4} = \sqrt[k_2]{\frac{\sum N_{obs,i}}{N_{obs,1}}} \qquad \text{(A106.5)}$$

where:

$N_{obs,i}$ number of lorries expected on lane i per year

$N_{obs,1}$ number of lorries on the slow lane per year

(108) The φ_{fat} value is a damage equivalent impact factor according to ENV 1991-3, Annex B:

$\varphi_{fat} = 1,2$ for surface of good roughness

$\varphi_{fat} = 1,4$ for surface of medium roughness

An additional impact factor for verification of sections within the distance of 6,0 m from an expansion joint should be taken into account according to ENV 1991-3, Annex B. (For more details see also ENV 1991-3, Annex B: Fatigue life assessment).

A106.3 Railway bridges

A106.3.1 Reinforcing and prestressing steel

P(101) The damage equivalent stress range for reinforcing and prestressing steel shall be calculated according to Equation (A106.6).

$$\Delta\sigma_{s,equ} = \lambda_s \; \Delta\sigma_{s,71} \qquad \text{(A106.6)}$$

where:

$\Delta\sigma_{s,71}$ steel stress range due to load model 71 (being placed in the most unfavourable position) under the infrequent combination of actions, which includes the dynamic factor according to ENV 1991-3.

λ_s correction factor to calculate the damage equivalent stress range from the stress range caused by load model 71. The values given in Table 106.2 are based on $\psi_1' = 1$.

P(102) The correction factor λ_s takes account of the span, annual traffic volume, service life and multiple tracks. It may be calculated from the following formula:

$$\lambda_s = \lambda_{s,1} \; \lambda_{s,2} \; \lambda_{s,3} \; \lambda_{s,4} \tag{A106.7}$$

where:

$\lambda_{s,1}$ factor to take account of the span of the member and the traffic mix
$\lambda_{s,2}$ factor to take account of the annual traffic volume
$\lambda_{s,3}$ factor to take account of the service life
$\lambda_{s,4}$ factor for multiple tracks.

(103) The factor $\lambda_{s,1}$ is a function of the span of the member and the traffic mix. The values of $\lambda_{s,1}$ for standard traffic mix and heavy traffic mix as defined in Tables F.1 and F.2 of ENV 1991-3 may be taken from Table A106.2 of this Appendix.
For other combinations of train types the factor $\lambda_{s,1}$ may be calculated from methods given in relevant documents[1].

(104) The $\lambda_{s,2}$ value denotes the influence of annual traffic volume and may be calculated from Equation (A106.8).

$$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 \times 10^6}} \tag{A106.8}$$

where:

Vol volume of traffic [tonnes per year and track].

k_2 slope of the S-N curve.

(105) The $\lambda_{s,3}$ value denotes the influence of the service life and may be calculated from Equation (A106.9).

$$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}} \tag{A106.9}$$

where:

N_{years} design working life of the bridge [years].

k_2 slope of the S-N curve.

(106) The $\lambda_{s,4}$ value denotes the effect of loading from more than one track. The effect of loading from two tracks may be calculated from Equation (A106.10).

$$\lambda_{s,4} = \sqrt[k_2]{n + (1-n)\; s_1^{k_2} + (1-n)\; s_2^{k_2}} \tag{A106.10}$$

$$s_1 = \frac{\Delta\sigma_1}{\Delta\sigma_{1+2}} \qquad s_2 = \frac{\Delta\sigma_2}{\Delta\sigma_{1+2}} \qquad n = \frac{N_c}{N_T}$$

where:

n proportion of traffic crossing simultaneously the bridge.

N_c number of trains crossing simultaneously the bridge.

N_T total number of trains running on one track.

[1] see, for example, the background Document *"Fatigue Design for Concrete Railway Bridges in Eurocode 2, Part 2, Loading, Resistance, Verification Formats"*.

$\Delta\sigma_1$, $\Delta\sigma_2$ stress range due to the load model 71 on one track.

$\Delta\sigma_{1+2}$ stress range due to the load model 71 on two tracks.

k_2 slope of the S-N curve.

If only compression stresses occur under traffic loads on a track, set the corresponding value $s_i = 0$.

Table A106.2: $\lambda_{s,1}$ values for single and continuous beams

a) simply supported beams

Type	S-N Curve			Span [m]	Traffic mix	
	k_1	k_2	N^*		Standard	Heavy
[1]	5	9	10^6	≤2	0,90	0,95
				≥20	0,65	0,70
[2]	3	7	10^6	≤2	1,00	1,05
				≥20	0,70	0,70
[3]	3	5	10^6	≤2	1,25	1,35
				≥20	0,75	0,75
[4]	3	5	10^7	≤2	0,80	0,85
				≥20	0,40	0,40

b) continuous beams (intermediate span, central section)

Type	S-N Curve			Span [m]	Traffic mix	
	k_1	k_2	N^*		Standard	Heavy
[1]	5	9	10^6	≤2	0,95	1,05
				≥20	0,50	0,55
[2]	3	7	10^6	≤2	1,00	1,15
				≥20	0,55	0,55
[3]	3	5	10^6	≤2	1,25	1,40
				≥20	0,55	0,55
[4]	3	5	10^7	≤2	0,75	0,90
				≥20	0,35	0,30

to continue

Table A106.2 (continued) : $\lambda_{s,1}$ values for single and continuous beams

c) continuous beams (end span section)

Type	S-N Curve			Span [m]	Traffic mix	
	k_1	k_2	N^*		Standard	Heavy
[1]	5	9	10^6	≤2	0,90	1,00
				≥20	0,65	0,65
[2]	3	7	10^6	≤2	1,05	1,15
				≥20	0,65	0,65
[3]	3	5	10^6	≤2	1,30	1,45
				≥20	0,65	0,70
[4]	3	5	10^7	≤2	0,80	0,90
				≥20	0,35	0,35

d) continuous beams (intermediate support section)

Type	S-N Curve			Span [m]	Traffic mix	
	k_1	k_2	N^*		Standard	Heavy
[1]	5	9	10^6	≤2	0,85	0,85
				≥20	0,70	0,75
[2]	3	7	10^6	≤2	0,90	0,95
				≥20	0,70	0,75
[3]	3	5	10^6	≤2	1,10	1,10
				≥20	0,75	0,80
[4]	3	5	10^7	≤2	0,70	0,70
				≥20	0,35	0,40

Definition of Types in Table A 106.2

[1] reinforcing steel, pre-tensioning (all), post-tensioning (strands in plastics and straight tendons in steel ducts).

[2] post-tensioning (curved tendons in steel ducts).

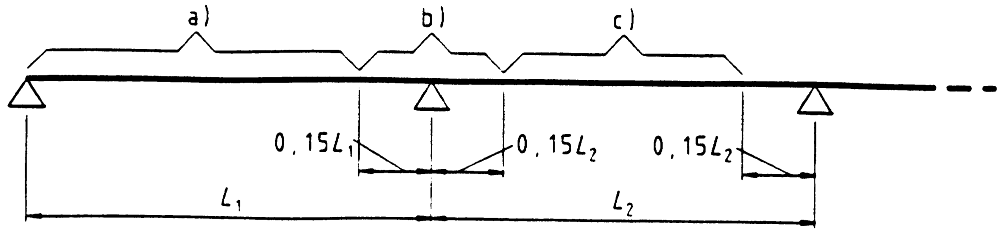
[3] couplers (prestressing steel).

[4] splicing devices (reinforcing steel), welded bars including tack welding and butt joints (see 5.2 of ENV 1992-1-1)

(107) Values of $\lambda_{s,1}$ for spans L between 2 m and 20 m may be obtained from the following equation:

$$\lambda_{s,1}(L) = \lambda_{s,1}(2) + [\lambda_{s,1}(20) - \lambda_{s,1}(2)] (\log L - 0,3)$$

For the definition of end span section, intermediate support section and intermediate span, central section see Figure A106.3 below.



a) end span section
b) intermediate support section
c) intermediate span, central section

Figure A106.3: Areas for span sections

A106.3.2 Concrete subjected to compression

(101) For concrete subjected to compression adequate fatigue resistance may be assumed if the following expression is satisfied:

$$14 \frac{1 - S_{cd,max,equ}}{\sqrt{1 - R_{equ}}} \geq 6 \qquad (A106.11)$$

where:

$$R_{equ} = \frac{S_{cd,min,equ}}{S_{cd,max,equ}} \quad ; \quad S_{cd,min,equ} = Sd \frac{\sigma_{cd,min,equ}}{f_{cd,fat}} \quad ; \quad S_{cd,max,equ} = Sd \frac{\sigma_{cd,max,equ}}{f_{cd,fat}}$$

$\sigma_{cd,max,equ}$ and $\sigma_{cd,min,equ}$ are the upper and lower stresses of the damage equivalent stress range with a number of cycles $N = 10^6$.

(102) The upper and lower stresses of the damage equivalent stress range should be calculated according to Equation (A106.12).

$$\sigma_{cd,max,equ} = \sigma_{c,perm} + \lambda_c \left(\sigma_{c,max,71} - \sigma_{c,perm}\right)$$
$$\sigma_{cd,min,equ} = \sigma_{c,perm} - \lambda_c \left(\sigma_{c,perm} - \sigma_{c,min,71}\right) \qquad (A106.12)$$

where:

$\sigma_{c,perm}$ compressive concrete stress under the infrequent combination of actions without load model 71.

$\sigma_{c,max,71}$, $\sigma_{c,min,71}$ maximum or minimum compressive stress under the infrequent combination of actions, which includes the dynamic factor ϕ_2 according to ENV 1991-3.

λ_c correction factor to calculate the upper and lower stresses of the damage equivalent stress range from the stresses caused by the load model 71.
The values given in Table 106.3 are based on $\psi_1' = 1$.

(103) The correction factor λ_c takes account of the permanent stress, the span, annual traffic volume, service life and multiple tracks. It may calculated from the following formula:

$$\lambda_c = \lambda_{c,0} \; \lambda_{c,1} \; \lambda_{c,2} \; \lambda_{c,3} \; \lambda_{c,4} \qquad \text{(A106.12)}$$

where:

$\lambda_{c,0}$ factor to take account of the permanent stress.

$\lambda_{c,1}$ factor to take account of the span of the member and the traffic mix.

$\lambda_{c,2}$ factor to take account of the annual traffic volume.

$\lambda_{c,3}$ factor to take account of the service life.

$\lambda_{c,4}$ factor for multiple tracks.

(104) The $\lambda_{c,0}$ value denotes the influence of the permanent stress and may be calculated from Equation (A106.14).

$$\lambda_{c,0} = 0{,}94 + 0{,}2 \frac{\sigma_{c,perm}}{f_{cd,fat}} \geq 1.0 \qquad \text{(A106.14)}$$

For the precompressed tensile zone in prestressed concrete members the value $\lambda_{c,0}$ may be taken equal to 1,0.

(105) The factor $\lambda_{c,1}$ is a function of the span of the member and the traffic mix. The values of $\lambda_{c,1}$ for standard traffic mix and heavy traffic mix as defined in Tables F.1 and F.2 of ENV 1991-3 may be taken from Table A106.3 of this Appendix.

(106) The $\lambda_{c,2}$ value denotes the influence of the annual traffic volume and may be calculated from Equation (A106.15).

$$\lambda_{c,2} = 1 + \frac{1}{8} \log \left[\frac{Vol}{25 \times 10^6} \right] \qquad \text{(A106.15)}$$

where: Vol denotes the volume of traffic [tonnes per year and track].

(107) The $\lambda_{c,3}$ value denotes the influence of the service life and may be calculated from Equation (A106.16).

$$\lambda_{c,3} = 1 + \frac{1}{8} \log \left[\frac{N_{years}}{100} \right] \qquad \text{(A106.16)}$$

where: N_{years} service life of the bridge [years].

(108) The $\lambda_{c,4}$ value denotes the effect of loading from more than one track. The effect of loading from two tracks may be calculated from Equation (A106.17).

$$\lambda_{c,4} = 1 + \frac{1}{8} \log n \geq 0{,}54 \quad \text{for } a \leq 0{,}8 \qquad \text{(A106.17)}$$

$$\lambda_{c,4} = 1{,}0 \quad \text{for } a > 0{,}8$$

$$a = \frac{\max \{\Delta\sigma_{c,1} , \Delta\sigma_{c,2}\}}{\Delta\sigma_{c,1+2}} \; ; \quad n = \frac{N_c}{N_T} \qquad \text{(A106.18)}$$

where:

n	proportion of traffic crossing the bridge.
N_c	number of trains crossing the bridge.
N_T	total number of trains running on one track.
$\Delta\sigma_{c,1}$, $\Delta\sigma_{c,2}$	compressive stress range caused by the load model 71 on one track.
$\Delta\sigma_{c,1+2}$	compressive stress caused by the load model 71 on two tracks.

Table A 106.3: $\lambda_{c,1}$ values for simply supported and continuous beams

a) simply supported beams

Zone of Cross section	Span [m]	Traffic mix	
		Standard	Heavy
compressed zone	≤ 2	0,70	0,70
	≥ 20	0,75	0,75
precompressed tensile zone	≤ 2	0,95	1,00
	≥ 20	0,90	0,90

b) continuous beams (intermediate span, central section)[1]

Zone of Cross section	Span [m]	Traffic mix	
		Standard	Heavy
compressed zone	≤ 2	0,75	0,90
	≥ 20	0,55	0,55
precompressed tensile zone	≤ 2	1,05	1,15
	≥ 20	0,65	0,70

c) continuous beams (end span section)

Zone of Cross section	Span [m]	Traffic mix	
		Standard	Heavy
compressed zone	≤ 2	0,75	0,80
	≥ 20	0,70	0,70
precompressed tensile zone	≤ 2	1,10	1,20
	≥ 20	0,70	0,70

d) continuous beams (intermediate support section)

Zone of Cross section	Span [m]	Traffic mix	
		Standard	Heavy
compressed zone	≤ 2	0,70	0,75
	≥ 20	0,85	0,85
precompressed tensile zone	≤ 2	1,10	1,15
	≥ 20	0,80	0,85

(108) Values of $\lambda_{c,1}$ for spans between 2 m and 20 m may be obtained from the following equation:

$$\lambda_{c,1}(L) = \lambda_{c,1}(2m) + [\lambda_{c,1}(20m) - \lambda_{c,1}(2m)]\ (\log L - 0{,}3)$$

[1] see Figure A106.3

APPENDIX 107 (informative)

Cable stayed bridges

A107.1 Introduction

(101) This Appendix applies to concrete cable stayed bridges designed in accordance with ENV 1992.

(102) The elements that constitute a stay cable are:
- the main tension elements (prestressing steel);
- the anchorages;
- the sheathing;
- the corrosion protection materials and devices.

(103) Only stay cables constituted by parallel wires, parallel strands or parallel bars are covered by this Appendix. Other types of stay cables are treated in ENV 1993-2.

A107.2 Prestressing steel

A107.2.1 Classification and geometry

(101) The certificate to each manufactured length of prestressing steel should contain as additional information the Guaranteed Ultimate Tensile Strength (GUTS), the tensile strength f_p and the modulus of elasticity.

(102) Prestressing steel for the fabrication of stay cables should be of the following types defined in EN 10138:

- Strands: stress relieved strands consisting of seven weldless cold drawn smooth wires;
- Wires: stress relieved, smooth, round cold drawn weldless wire;
- Bars: hot rolled, with thermal or mechanical treatments.

A107.2.2 Ductility characteristics

(101) Strands should have the following D-values according to Annex 1 of EN 10138-1:

- $D = 28$ for strands with nominal diameters between 12,5 and 13 mm;
- $D = 20$ for strands with nominal diameters between 15,2 and 16 mm.

A107.2.3 Fatigue

(101) Fatigue tests in accordance with EN 10138 or other relevant standards should be performed on samples taken from each manufactured length of prestressing steel.

(102) The material of the cables should withstand without failure $2x10^6$ cycles of stress fluctuating down from a maximum stress of $0,8\,f_{pk}$., where f_{pk} is the characteristic tensile strength of the steel used.

The fluctuating stress ranges are:

- for strands $\Delta\sigma_p = 195$ N/mm²
- for wires $\Delta\sigma_p = 200$ N/mm²
- for smooth bars $\Delta\sigma_p = 200$ N/mm²
- for ribbed bars $\Delta\sigma_p = 180$ N/mm²

A107.2.4 Susceptibility to stress corrosion

(101) Adequate resistance to stress corrosion should be demonstrated by performing tests as defined in EN 10138.

A107.3 Anchorages, saddles and couplers

(101) The design of the anchorages and the saddles should provide for removal and replacement of the stay cable and for future adjustments of forces.

(102) Anchorages, saddles and couplers should not fail before the failure of the main tension elements.

A107.4 Sheating for stay cables

(101) Sheathing for stay cables may be of any of the following types:

- steel pipe, or polyethylene pipe, used to enclose the complete tension elements;
- polyethylene sheathing extruded directly over the individual strands.

(102) The pipe wall thickness should be sufficient to withstand all stresses during handling, grouting and service.

(103) All pipe welds should be capable of developing the full yield strength of the pipe cross section and have sufficient fatigue resistance.

(104) Steel tubes should be, at least, of the quality Fe 510 according to EN 10025:1990 and should comply with the relevant standards.

(105) Welding of steel pipes should not be permitted with the tendons inside the pipes.

(106) All field welds on steel pipes should be verified by ultrasonic and/or radiographic testing.

(107) The polyethylene pipe, or individual sheath, should be made of high density polyethylene (HDPE) complying with ISO 161/1 and ISO 3607, or other relevant standards.

(108) For polyethylene pipes the maximum ratio of outside diameter to minimum wall thickness should be |18|.

(109) The minimum thickness of HDPE sheathing extruded directly over individual strands should be |1,5| mm.

A107.5 Corrosion protection system

(101) Adequate corrosion protection of stay cables should be provided.

(102) The tightness and drainage of boots should be assured to avoid water entering and/or accumulating in the recess tubes.

A107.6 Arrangement of stay cables

(101) Stay cables should be arranged such, that bending stresses in the stay cables at anchorages are avoided.

(102) In the area of the deck stay cables should be protected against damage caused by vehicle collision and/or by the users.

A107.7 Fabrication of stay cables

(101) The procedures described in 6.3.4.2 of ENV 1992-1-1 should be strictly observed.

(102) During fabrication of stay cables it should be ensured that all tendons are installed parallel to each another.

(103) When handling prefabricated cables it should be ensured that the bending radius is never less than |25| times the diameter of the cable sheath.

(104) Any strand, wire or bar damaged during fabrication or handling should be replaced, and damaged protection should be repaired or replaced.

A107.8 Installation of stay cables

(101) Installation and tensioning of each stay cable should follow a detailed programme prescribing the cable elongation and/or the cable forces, acceptable tolerances and deck and pylon displacements.

(102) Any significant deviation from the assumed construction procedures and tolerances, ambient temperature, loads or displacements, should immediately lead to a complete revision of the cable installation programme.

(103) Permanent monitoring of the stay's installation including survey of the deck and the pylon alignment and ambient temperature should be conducted and recorded.
A complete record of the final forces, the final levelling and the ambient temperature during the measurements should be made.

(104) Any deviation of more than |±2,5%| from the final stay forces predicted in the installation program should be investigated and corrected.

(105) The amplitude of movement to be allowed in the anchorage adjusting devices, should be previously specified.
Any necessary adjustment corresponding to a reduction of force on any stay should be possible without the need to release any of the internal stay anchorage component devices.

A107.9 Grouting operations

(101) In addition to the rules stated in 6.3.4.6 of ENV 1992-1-1, where relevant, stay cable grouting operations should comply with the following.

(102) Grout lifts in HDPE pipe sheathing should not exceed |35| metres in height. Each first grout lift should not exceed |6| metres.

(103) The last lift should be small enough to minimise the possibility of the creation of a void at the top of the stay.

(104) Diametrical expansion of the pipe at the injection point should not exceed |2%| of the original diameter. Before proceeding with a new grouting lift the pipe should be tightened against the hardened grout at the top of the previous grout lift.

A107.10 Acceptance tests for stay cables

(101) Adequate fatigue strength of the stay cables proposed for each project should be demonstrated by suitable acceptance tests.
The specimens should incorporate the same coupling devices, grouting conditions and sheathing type as the final system.

(102) Tests should reproduce any flexural effect or transverse stress expected to occur in the stays.

(103) Previous tests conducted on specimens similar in every aspect to the recommended stay cables for a new project may be accepted for approval.

(104) The proposed stay cables should conform to the following minimum performance requirements:

- The stay cable should withstand |2×10^6| cycles of undulating stress for an upper stress of |0,45| f_{pk} and a stress range of |200| N/mm².
- During the test no more than |2%| of the prestressing steel cross section should fail and the anchorage components should not show any anomaly.
- After the completion of the fatigue test the specimen should be loaded in increments and should withstand a minimum of |0,90| f_{pk} without the failure of any of its components.

A107.11 Basis of design

A107.11.1 General

(101) Thermal actions should be taken into account for the serviceability limit state and fatigue limit state verifications. They should include temperature differentials between stays (taking into consideration the colour of the stays) and deck and pylons, including thermal gradients through the deck and the pylons.

A107.11.2 Ultimate limit states

(101) The tensile resistance of the stays should be checked under the combination of actions corresponding to the ultimate limit state with a partial safety factor of γ_s = |1,5| applied to the characteristic tensile strength, f_{pk}, of the prestressing steel.

A107.11.3 Serviceability limit states

(101) Under the frequent combination of actions the tensile stress in the stays should not exceed |0,45| f_{pk}.

A107.11.4 Ultimate limit state of fatigue

(101) Fatigue failure of stay systems is usually governed by local effects at anchorages, saddles and coupling devices, and, where possible, verification should be performed by means of test results.

(102) Except for footbridges, the main tension elements of the stays should be checked with respect to the resistance to fatigue.

(103) The verification for the ultimate limit state of fatigue should be performed with the same combinations of actions used for the check of the fatigue resistance of the other members of the bridge.

(104) The stress fluctuation in the stays under the load combination relevant for fatigue should include the associated flexural stress due to the rotations of the anchorages.

(105) Fatigue verification of the prestressing steel should be carried out according to 4.3.7.5 of this Part 2.

A107.12 Limitation of damage due to accidental action

(101) The danger of collapse of the structure, due to the failure of one or more stays, under an accidental action such as impact, fire or explosion of a vehicle should be assessed.

(102) If not stated otherwise, the Application Rules below may be used.

(103) It should be checked that, in the event of a failure of all stays in one row of stays within any |20| m length of the bridge, the bridge will not collapse under the accidental combination of actions using a partial safety factor of γ_s = 1,3 for the prestressing steel.

(104) The design should account for the temporary loss of any one stay without the need to reduce the traffic loads during the stay repair period.

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