

Eurocode 4: Design of composite steel and concrete structures —

Part 2: Composite bridges

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

ICS 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 Constructional Steelwork Association
British Precast Concrete Federation
Concrete Society
County Surveyors' Society
Department of the Environment, Transport and the Regions (DETR) —
Highways Agency
Institution of Civil Engineers
Institution of Structural Engineers
Railtrack plc
Steel Construction Institute
UK Steel Association
Welding Institute

This Draft for Development, having been prepared under the direction of the Sector Policy and Strategy Committee for Building and Civil Engineering, was published under the authority of the Standards Policy and Strategy Committee on 17 January 2002

© BSI 17 January 2002

Amendments issued since publication

Amd. No.	Date	Comments

The following BSI reference relates to the work on this Draft for Development:
Committee reference B/525/10

Contents

	Page
Committees responsible	Inside front cover
<u>National foreword</u>	<u>ii</u>
Text of National Application Document	iii
<u>Text of ENV 1994-2</u>	<u>1</u>

National foreword

This Draft for Development has been prepared by Subcommittee B/525/10 and comprises the English language version of ENV 1994-2:1997, *Eurocode 4: Design of composite steel and concrete structures — Part 2: Composite bridges*, as published by the European Committee for Standardization (CEN), and the National Application document (NAD) for use with the ENV for the design of bridges to be constructed in the United Kingdom (UK).

ENV 1994-2:1997 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structure and design of building and civil engineering works.

This publication is not to 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 (EN).

The values for certain parameters in the ENV Eurocodes may be set by CEN members to conform to the requirements of national regulations. These parameters are designated by □ (boxed values) in the ENV. The values to be used in the UK are tabulated in this NAD.

During the ENV period of validity, reference should be made to the supporting documents listed in the NAD.

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

The Building Regulations 1991, Approved Document A 1992 (published December 1991) [1], draws designers' attention to the potential use of ENV Eurocodes as an alternative approach to Building Regulation compliance. ENV 1994-2:1997 has been thoroughly examined over a period of several years and is considered to offer such an alternative approach when used in conjunction with this NAD.

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

Comments should be sent in writing to BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, proposed revision, within two years of the issue of this document.

Compliance with ENV 1994-2:1997 and this NAD does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, and inside front cover, pages i to xvii, a blank page, an inside back cover, the ENV title page, pages 2 to 98, 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 1994-2:1997

**Contents of
National Application Document**

	Page
Introduction	v
1 Scope	v
2 Normative references	v
3 Partial safety factors, combination factors and other values	vi
4 Loading codes	viii
5 Reference standards	viii
<u>6 Additional Recommendations</u>	<u>xi</u>
Table 1 — Values to be used in referenced clauses instead of boxed values	vii
Table 2 — References in ENV 1994-2 to other publications, together with additional relevant documents	ix
<u>Bibliography</u>	<u>xvii</u>

Introduction

This National Application Document (NAD) has been developed from:

- a) a textual examination of ENV 1994-2:1997;
- b) a calibration against UK practice, supporting standards and test data;
- c) trial calculations.

1 Scope

This NAD provides information required to enable ENV 1994-2:1997 (hereafter referred to as EC4-2) to be used with ENV 1994-1-1:1992, as qualified by its NAD, for the design of bridges to be constructed in the UK. Where there is conflict between the NADs for ENV 1994-1-1 and ENV 1994-2, the NAD for ENV 1994-2 is to govern.

2 Normative references

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

BS EN ISO 13918, *Welding — Studs and ceramic ferrules for arc stud welding*.

BS EN ISO 14555, *Welding — Arc stud welding of metallic materials*.

ENV 1991-1, *Eurocode 1: Basis of design and actions on structures — Part 1: Basis of design*.

ENV 1991-2-1, *Eurocode 1: Basis of design and actions on structures — Part 2-1: Actions on structures — Densities, self-weight and imposed loads*.

ENV 1991-2-3, *Eurocode 1: Basis of design and actions on structures — Part 2-3: Actions on structures — Snow loads*.

ENV 1991-2-4, *Eurocode 1: Basis of design and actions on structures — Part 2-4: Actions on structures — Wind actions*.

ENV 1991-2-6, *Eurocode 1: Basis of design and actions on structures — Part 2-6: Actions on structures — Actions during execution*.

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

ENV 1998-2, *Eurocode 8: Design provisions for earthquake resistance of structures — Part 2: Bridges*.

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

ENV 1090-5, *Execution of steel structures — Part 5: Supplementary rules for bridges*.

ENV 1991-2-5, *Eurocode 1: Basis of design and actions on structures — Part 2-5: Actions on structures — Thermal actions*.

ENV 1991-2-7, *Eurocode 1: Basis of design and actions on structures — Part 2-7: Actions on structures — Accidental actions due to impact and explosions*.

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

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

DD ENV 1994-2:2001

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

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

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

ENV 1993-1-1, *Eurocode 3: Design of steel structures — Part 1-1: General rules and rules for buildings.*

ENV 1993-1-3, *Eurocode 3: Design of steel structures — Part 1-3: General rules — Supplementary rules for cold formed thin gauge members and sheeting.*

ENV 1993-1-5, *Eurocode 3: Design of steel structures — Part 1-5: General rules — Supplementary rules for planar plated structures without transverse loading.*

ENV 1993-2, *Eurocode 3: Design of steel structures — Part 2: Steel bridges.*

ENV 1994-1-1, *Eurocode 4: Design of composite steel and concrete structures — Part 1.1: General rules and rules for buildings.*

ENV 10080, *Steel for the reinforcement of concrete — Weldable ribbed reinforcing steel B500 — Technical delivery conditions for bars, coils and welded fabric.*

ISO 3898, *Bases for design of structures — Notations — General symbols.*

3 Partial safety factors, combination factors and other values

- a) The values for combination factors (ψ) for bridges should be those given in Tables 3 and 4 of the NAD for use with ENV 1991-3:1994.
- b) The values for partial safety factors (γ) should be those given in ENV 1994-1-1 and ENV 1994-2, except as modified by the relevant UK NADs.
- c) Other values should be those given in ENV 1994-2 as modified by Table 1 of this NAD.

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

Reference in ENV 1994-2	Definition	Symbol	Condition	Value	
				Boxed EC4.2	UK NAD
2.3.3.1(5)	Partial safety factor for prestress imposed by jacking	γ_p	Ultimate	1.0	1.0
2.3.3.1(6)	Partial safety factor for nominal free shrinkage		Ultimate	1.0	1.0
Table 2.3	Partial safety factors for resistances and material properties for — structural steel — structural steel Class 4 compression — concrete — steel reinforcement and prestressing tendons — profiled steel sheeting	γ_a γ_{Rd} γ_c γ_s γ_{ap}	Ultimate fundamental	1.0 1.10 1.5 1.15 1.10	1.0 1.10 1.5 1.15 1.10
Table 2.3	Partial safety factors for resistances and material properties for — structural steel — structural steel Class 4 compression — concrete — steel reinforcement and prestressing tendons — profiled steel sheeting	γ_a γ_{Rd} γ_c γ_s γ_{ap}	Ultimate accidental	1.0 1.0 1.3 1.0 1.0	1.0 1.0 1.3 1.0 1.0
2.3.4(6)	Partial safety factor for materials at serviceability	γ_M	Serviceability	1.0	1.0
4.4.1.4(3)	Partial safety factor for structural steel in compression for Class 4 sections	γ_{Rd}	Ultimate	1.10	1.10
4.7.2(6)	Additional partial safety factor on shear connectors	γ_F	Ultimate	1.25	1.25
5.2(2)	Factor of $f_c(t)$	—	Serviceability	0.6	0.6
5.2(3)	Factor of f_{ck}	—	Serviceability	0.6	0.6
5.2(5)	Factor of f_{sk}	—	Serviceability	0.8	0.8
5.2(6)	Factor of f_{pk}	—	Serviceability	0.65	0.65
5.2(7)	Factor of $f_c(t)$	—	Serviceability	0.6	0.6
5.2(7)	Limit by which $0.6 f_c(t)$ may be exceeded during construction	—	Serviceability	10 %	10 %
6.1.5(4)	Partial safety factor for stud shear connectors	$\gamma_{Mf,v}$	Fatigue	1.0	1.0
6.3.2.1(1)	Partial safety factors for headed studs	γ_v	Ultimate	1.25	1.25

4 Loading codes

The loading codes to be used are (as qualified by the relevant UK NADs):

ENV 1991-1, *Eurocode 1: Basis of design and actions on structures — Part 1: Basis of design.*

ENV 1991-2-1, *Eurocode 1: Basis of design and actions on structures — Part 2-1: Densities, self-weight and imposed loads.*

ENV 1991-2-3, *Eurocode 1: Basis of design and actions on structures — Part 2-3: Snow loads.*

ENV 1991-2-4, *Eurocode 1: Basis of design and actions on structures — Part 2-4: Wind loads.*

ENV 1991-2-6, *Eurocode 1: Basis of design and actions on structures — Part 2-6: Loads and deformations imposed during execution.*

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

ENV 1998-2, *Eurocode 8: Design provisions for earthquake resistance of structures — Part 2: Bridges.*

ENV 1991-2-5, *Eurocode 1: Basis of design and actions on structures — Part 2-5: Thermal actions.*

ENV 1991-2-7, *Eurocode 1: Basis of design and actions on structures — Part 2-7: Accidental actions due to impact and explosions.*

Where any of the above codes have not been published, the UK supporting standard should be used as indicated in clause 5.

5 Reference standards

Standards including materials specifications and standards for construction are listed for reference purposes in Table 2. Published ISO and CEN standards are not included in Table 2.

Table 2 — References in ENV 1994-2 to other publications, together with additional relevant documents

Reference in EC4-2	Document referred to	Document title or subject area	Status	UK Document	DMRB Document ^a
1.1.2(6), 1.4.3(3), 1.6.1(1), 1.7(3), 2.2.1.1(4), 2.3.2.4(1), 2.3.3.2(5), 2.3.4(6), 2.4(4), 3.3.1(1), 3.3.1(2), 3.3.2(1), 3.3.5(1), 3.3.6(1), 4.1.1(2), 4.1.1(3), 4.4.2(1), 4.4.3(2), 4.6.2(1), 4.6.3(1), 4.6.3(2), 4.11(1), 4.11(3), 4.12.1(3), 4.12.2(2), 4.12.2(3), 4.12.2(4), 4.12.3(4), 4.12.5(3), 4.12.6(1), 5.2(8), 5.5(1), 6.1.5(2), 6.1.5(4), 6.1.5(5), 6.1.5(7), 6.5(1), K.4.4(1)	ENV 1993-2	Steel bridges	Published 1997	BS 5400-3	BD 13 [2]
1.1.2(8)	ENV 1090-5	Fabrication and erection of steel structures	Draft	BS 5400-6	Specification for Highway Works [3]
1.3(1), 2.3.3.2(3), 2.3.4(7), 3.1.2(1), 3.1.2(3), 3.1.3(1), 3.1.3(2), 3.1.4(1), 3.1.4.2(1), 3.1.5(1), 3.2.1(1), 3.2.2(1), 3.2.4(1), 3.6(1), 4.2.1(3), 4.2.2.3(1), 4.2.3(4), 4.4.1.3(3), 4.4.1.3(4), 4.4.1.4(3), 4.5.3.5(1), 4.7.2(1), 4.7.2(4), 4.7.2(5), 4.11(2), 5.2(7), 5.3.1(6), 5.3.2.2(1), 6.4.5(1), 6.6.2(1), 6.6.4(1), 6.6.5, 7.7.2(3), 7.7.3(2), 9.4.3.2(1), 9.4.3.2(2)	ENV 1992-1-1	Concrete structures	Published 1991	—	—
1.3(1), 1.4.3(4), 1.6.7(1), 1.6.8(1), 3.2.4(1), 4.3.2(2), 4.4.1.1(6), 4.4.3(1), 6.1.5(7), 6.5(1), 9.4.3.2(1), 9.4.3.3(2), 10.1(1), 10.1(2)	ENV 1993-1-1	Steel structures	Published 1992	—	—
1.4.2.7, 1.4.3(6), 2.3.3.2(4), 3.4(1), 4.6.2(2), 4.8.1(4), 6.3.2.1(2), 6.3.3(1), 6.4.3(1), 6.4.5(2), 6.6.3(1), 9.4.4(1), 10.3(1)	ENV 1994-1-1	Composite steel and concrete structures	Published 1994	—	—
1.4.3(1), 2.2.2.3(4), 2.2.5(1), 2.3.2.2(1), 2.3.3.1(3), 2.3.4(2), 4.5.3.4(5), 4.12.2(1), 4.12.2(2), 4.12.3(2), 4.12.4(7), 5.1.4.1(4), 5.4(5), 5.5(1), 6.1.5(6)	ENV 1991-3	Traffic loads on bridges	Published 1995	BS 5400-2	BD 37 [4]
^a DMRB Design Manual for Roads and Bridges.					

Table 2 — References in ENV 1994-2 to other publications *(continued)*

Reference in EC4-2	Document referred to	Document title or subject area	Status	UK Document	DMRB Document ^a
1.4.3(2), 1.6.1(1), 1.7(3), 2.2.1.1(4), 2.3.3.1(4), 2.4(3), 3.1.3(3), 4.1.1(2), 4.1.1(10), 4.5.3.4(3), 4.7.2(1), 4.8.2.5(2), 4.8.3.13(6), 4.12.1(2), 4.12.2(2), 4.12.2(5), 4.12.3(3), 4.12.5(1), 4.12.5(2), 4.12.6(1), 4.12.6(3), 4.12.6(4), 5.1.2(1), 5.1.2(2), 5.1.2(3), 5.2(2), 5.2(3), 5.2(5), 5.2(6), 5.2(7), 5.3.1(2), 5.3.1(7), 5.3.2.1(1), 5.3.2.2(2), 5.5(1), 6.4.1.2(2), 8.1(2), K.2(3), K.4.2(2), K.4.3(1), K.4.3(2), K.5.1(1), K.5.3(1), K.5.4(1), L.5.2(3)	ENV 1992-2	Concrete bridges	Published 1996	BS 5400-4	BD 24/92 [5]
1.4.3(5), 1.6.1(1), 2.2.1.1(4), 4.2.1(6), 4.2.2.1(2), 4.2.2.2(1), 4.2.2.2(6), 4.2.2.3(2), 4.3.1(11), 4.4.1.2(5), 4.4.1.4(1), 4.4.2(1), 4.4.3(3), 4.4.5, 4.4.6(1)	ENV 1993-1-5	Steel plated structures	Published 1997	BS 5400-3	BD 13/90 [2]
1.6.1(1)	ISO 3898	Basis for design of structures: general symbols	Published 1997	—	—
1.7(3), 2.2.5, 2.3.3.2(6), 2.3.4(2), 10.1(2)	ENV 1991-1	Basis of design	Published 1994	BS 5400-1	BD 15/92 [6]
1.7(3), 6.3.2.1(1), 9.4.3.1(1)	EN ISO 14555	Arc stud welding	Published 1998	BS EN ISO 14555	—
1.7(3), 3.5.2(7), 6.3.2.1(2), 9.4.3.1(1)	EN ISO 13918	Welding studs for arc stud welding	Published 1998	BS EN ISO 13918	—
2.2.2.2(7), 2.2.2.3(4)	ENV 1991-2-6	Loads and deformations during erection	Draft	BS 5400-2	BD 37/88 [4]
2.3.3.1(1)	ENV 1991	Basis of design and actions on structures	Various	BS 5400-2	BD 37/88 [4]
3.1.2(4), 3.1.2(5), 3.1.4.1(3), 3.1.4.2(1), 6.3.8(3)	ENV 1992-1-4	Lightweight concrete structures	Published 1996	—	—
^a DMRB Design Manual for Roads and Bridges.					

Table 2 — References in ENV 1994-2 to other publications (*concluded*)

Reference in EC4-2	Document referred to	Document title or subject area	Status	UK Document	DMRB Document ^a
3.1.3(2)	ENV 206	Concrete classes	Published 1990	—	—
3.1.6(1), 5.1.3(8), 6.2.5(1)	ENV 1991-2-5	Thermal actions	Draft	BS 5400-2	BD 37/88 [4]
3.2.2(1)	ENV 10080	Ribbed bars or wires	Published 1995	—	—
4.5.3.5(1)	ENV 1992-1-6	Plain concrete structures	Published 1994	—	—
4.11(5), 7.1.1(5)	ENV 1993-1-3	Cold formed thin gauge members and sheeting	Draft	—	—
8.4(3)	ENV 1992-1-3	Precast concrete	Published 1994	—	—
(ENV 1994-1-1) 9.1(4), ENV note at the end of 9.1	Reference standards or other documents	Responsibility or other requirements to the contractor	—	Contractor's obligations stated in contract documents, e.g. drawings specification bills of quantities contract	Specification for Highway Works [3]
^a DMRB Design Manual for Roads and Bridges.					

6 Additional recommendations

6.1 Guidance on EC4-2

NOTE 6.1.1 to 6.1.11 should be followed when designing in accordance with EC 4-2.

6.1.1 Foreword

a) Paragraph (18)

The relevant authorities are the Technical Approval Authorities such as the Highways Agency, Highways Directorate of the National Assembly for Wales, Scottish Executive National Roads Directorate, Northern Ireland Department of Environment, Local Authorities (County Council Highways Departments etc.), Railtrack, London Underground, British Waterways Board, Docklands Light Railway, etc.

b) Paragraph (20)

All references to ENV 1994-1-1 in EC4-2 should be interpreted as being to ENV 1994-1-1 as qualified by its UK NAD.

6.1.2 Section 1. General

- a) Clause **1.1.2(2)P**
Unless otherwise agreed by the relevant authorities, the relevant clauses of ENV 1994-2 may be used for the design of cable stayed bridges and bridges incorporating unbonded tendons.
- b) Clause **1.2(5)**
Alternative design rules different from the Application Rules should not be used without the specific approval of the relevant authority.
- c) Clause **1.7(2)**
All references to any ENV should be interpreted as being to that ENV as qualified by its UK NAD.

6.1.3 Section 2. Basis of design

- a) Clause **2.2.1.2(2)**
Strength and stability should be checked for the construction stage where the partially constructed beam (particularly a steel beam acting non-compositely) supports the formwork and the imposed load of fresh concrete plus construction loads or temporary storage loads.

6.1.4 Section 3. Materials

- a) Clause **3.3.1(1)**
S420 and S460 steel should not be used in composite compression members.
- b) Clause **3.4(1)**
Profiled steel sheeting should not be used for composite slabs without the specific approval of the relevant authority. Profiled steel sheeting may be used as non-participatory permanent formwork.

6.1.5 Section 4. Ultimate limit states

- a) Clause **4.1.1(10)**
In most cases, the width of the local actions (being approximately equal to the width of a vehicle) will be less than the effective width and therefore not “comparable”.

Combinations of these global and local effects will therefore generally not need to be considered.
- b) Clause **4.2.2.2(2)**
For other types of shear connector, b_0 is measured from the edge of the connector.
- c) Clause **4.3.3(1)**
This clause defines the stress distribution to be used in determining the classification of webs, particularly at the boundaries between classes. Plastic distribution should be used for the Class 1/2 and Class 2/3 boundaries and elastic distribution for the Class 3/4 boundary.
- d) Clause **4.4.1.4(3)P**
Where the compression flange of structural steel is in an effective cross-section in Class 4 and is adequately restrained by shear connection to a reinforced concrete deck slab, the limiting stress may be taken as f_y/γ_a .
- e) Clause **4.4.2(1)P**
A contribution from the reinforced concrete part of the beam to the resistance to vertical shear should not be considered without the specific approval of the relevant authority.
- f) Clause **4.5.3.4**
For the majority of bridges, the methods given in paragraphs (3) and (4) will be appropriate.

- g) Clause **4.5.4(1)**
Non-linear global analysis which models the non-linear behaviour caused by yielding of structural steel, reinforcement or prestressing steel should not be used without the specific approval of the relevant authority.

In addition to the effects listed, consideration should also be given to the non-linear behaviour of concrete under load and the maximum allowable fabrication and construction tolerances of the composite member and its elements.
- h) Clause **4.6.2(2)**
Reference may also be made to annex F of ENV 1993-1-1:1992 in order to calculate M_{cr} .
- i) Clause **4.8.1(4)**
S420 and S460 steel should not be used in composite compression members.
- j) Clause **4.8.2.7(2)**
The design shear strength due to bond and friction for completely concrete encased sections should be taken as 0.4 N/mm^2 .
- k) Clause **4.8.3.13(8)**
Typographical error: **4.8.3.11(3)** for γ_{Ma} should read **4.8.3.2(1)**.
- l) Clause **4.12.3(3)**
The factored load model is only used for the verification of concrete and reinforcing and prestressing steel.
- m) Clauses **4.12.4, 4.12.6**
For the majority of bridges, the simplified method of assessment in **4.12.6** will be appropriate using the simplified methods of calculating the stress range $\Delta\sigma_E$ given in **4.12.4**; i.e. there will be no need to use annex L.
- n) Clause **4.12.4(8)**
 $M_{min,f}$ in last line should read $M_{min,F,E}$
- o) Clause **4.12.5(1)**
This generally means that no fatigue verification for concrete is required; see **5.2(3)** and ENV 1992-2:1996, **4.3.7.4** and Figure 4.134.
- p) Clause **4.12.6(3)**
Reference should also be made to ENV 1992-2:1996, **4.3.7.1 (102)**, as amended by its NAD, which lists circumstances where a fatigue verification is generally not necessary.

6.1.6 Section 5. Serviceability limit states

- a) Clause **5.1.2(2)**
For the appropriate design category, reference should be made to the NAD for ENV 1992-2.
- b) Clause **5.1.4.1(1)**
Torsional warping, distortional warping and distortional bending stresses should be considered in the design of box girders.
- c) Clause **5.1.4.1(4)**
A combination factor $\psi = 1$ means that the stresses due to global and local effects are added without any reduction.
- d) Clause **5.3.1(2)**
Reference should be made to the NAD for ENV 1992-2 for the appropriate design category to be used.
- e) Clause **5.3.1(4)**
Design crack width limits should either be in accordance with **5.3.1(6)**, **5.3.1(7)**, **5.3.2** and **5.3.3**, agreed with the client or as specified in the NAD to ENV 1992-2.

6.1.7 Section 6. Shear connection**a) Clause 6.2.4.3(5)**

The maximum shear force equation given may be used for stud shear connectors. For other types of connector, the maximum shear force per unit length is given by:

$$V_{d,max} = 2V_1(e_d + b_{eff} / 2)$$

b) Clause 6.3.2.1

The following text is intended as a note to **6.3.2.1**.

NOTE The minimum ultimate tensile strength of the material of a headed stud is usually specified in the UK as 495 N/mm².

c) Clause 6.3.2.2(2)

Where the force $F_{t,Sd}$ exceeds 0.1 P_{Rd} , the following interaction criterion may be used for normal density concrete:

$$(T_u / T_{Rd})^{5/3} + (P_u / P_{Rd})^{5/3} \leq 1$$

where

T_u = tension applied to stud at ultimate;

P_u = shear applied to stud at ultimate;

$$T_{Rd} = 1.33h (h/d + 1.5) (f_{ck}')^{1/2} d/\gamma_v \leq (\pi/4)f_u d^2/\gamma_v.$$

T_u should not exceed 0.5 T_{Rd} . Attention should be given to ensuring that tension on a group of studs does not result in a group failure and that the tension can be satisfactorily distributed into the section.

d) Clause 6.3.4(5)

The leg length of the weld between a block connector and the plate to which it is attached should not exceed half the thickness of the plate.

e) Clause 6.3.6(3)

The leg length of the weld between a block connector with anchor or hoop and the plate to which it is attached should not exceed half the thickness of the plate.

f) Clause 6.3.7(3)

The leg length of the weld between an angle connector and the plate to which it is attached should not exceed half the thickness of the plate.

g) Clause 6.5(1)

Friction grip bolts as shear connection should not be used without the specific approval of the relevant authority.

h) Clause 6.5(2)

For the coefficient of friction to be used between steel and concrete, see **6.1.9a)** of this NAD.

i) Clause 6.5(3)

The category of bolted connection in accordance with ENV 1993-1-1:1992, **6.5.3.1** may usually be taken as Category B, i.e. no slip at serviceability.

j) Clause 6.5(4)

Detailing of friction grip bolts should be in accordance with ENV 1994-1-1:1994, **6.5.4**.

k) Figure 6.12

In the diagram showing potential surface of shear failure type c-c, the hoop should have been shown extending above the A_b reinforcement in order to give a 30 mm clear distance in accordance with ENV 1994-1-1:1994, **6.4.1.1**.

l) Clause **6.6.2(2)**

In beams where the design loading causes transverse sagging bending in the slab in the region of the shear connectors, account should be taken of the effect of this where the shear plane does not cross the whole depth of the slab but does intersect with the reinforcement required to resist the tension caused by this sagging bending (types b-b and c-c in Figure 6.12 but not a-a and d-d).

The area of reinforcement A_e should be taken as:

$$A_e = 2 A_{\text{tension}}$$

where

$$A_{\text{tension}} = \text{area of reinforcement required to resist tension}$$

m) Clause **6.6.2(2)**

In beams where the design loading causes transverse tension in the slab (for example, transverse composite beams in the deck of a composite tied arch), consideration should be given to the effect of this tension in calculating the area of reinforcement available to resist longitudinal shear for shear failure types a-a, b-b, c-c and d-d in Figure 6.12.

6.1.8 Section 7. Composite slabs with profiled steel sheeting and composite plates

a) Clause **7.1.1(5)**

Profiled steel sheeting should not be used for composite slabs without the specific approval of the relevant authority. If profiled steel sheeting is used, the applicability of **7.1** to **7.6** as well as **3.4**, **4.2.1(4)**, **6.3.3**, **6.4.3**, **6.6.3**, **9.4.4**, **10.3** and annex E should be agreed with the relevant authority.

Profiled steel sheeting may be used as non-participatory permanent formwork for slabs.

6.1.9 Section 8. Decks with precast concrete slabs

a) Clause **8.4(2)**

For steel surfaces blasted with shot or grit and either spray-metallized with aluminum or with loose rust removed and no pitting, the friction coefficient between slab and steel beam may be taken as 0.5.

6.1.10 Section 9. Execution

a) Clause **9.3**

In particular, consideration should be given to:

i) The stability of non-composite steelwork, see ENV 1994-1-1:1994, **9.3** and **6.1.3a**) of this NAD.

ii) The out-of-balance loading that may occur during concreting or the placing of precast slabs. If the concrete is placed in the bay on one side of a beam and not on the other, a torsional load may be applied to the beam which could affect its stability.

6.1.11 *Annex C*

a) Clause **C.6.2(1)**

In equation C.10, $(h - 2t_f)t_w^2$ should read $(h - 2t_f)^2 t_w$

6.1.12 *Annex K*

a) Clause **K.4.3(1)**

A contribution from the reinforced concrete part of the beam to the resistance to vertical shear should not be considered without the specific approval of the relevant authority.

6.1.13 *Annex L*

- a) This annex will not generally need to be used for normal design purposes. Most clauses in the code give alternative methods that do not require the use of this annex.

6.2 Recommendations on subjects not covered in EC4-2

6.2.1 *Design resistance of channels as shear connectors*

The design resistance of a channel connector may be determined from the following (assuming the web of the channel is vertical with the shear applied nominally perpendicular to the web):

$$P_{Rd} = 20 b h^{3/4} f_{ck}^{1/3} / \gamma_v$$

where

- P_{Rd} is in newtons (N);
 b is the length of the channel in millimetres (mm);
 h is the height of the channel in millimetres (mm);
 f_{ck} is the characteristic cylinder strength of concrete in newtons per millimetres squared (N/mm²).

The partial safety factor γ_v should be taken as 1.25 for the ultimate limit state.

The recommendations for channel dimensions are as follows:

- The height h of the channel should not exceed 20 times web thickness nor 150 mm.
- The width b of the channel should not exceed 300 mm.
- The underside of the top flange of the channel should not be less than 30 mm clear above the bottom reinforcement.
- The leg length of the weld connecting the channel to the plate should not exceed half the plate thickness.

Use of a channel section may be deemed to prevent uplift of the concrete slab in accordance with ENV 1994-2:1997, **6.1.1(4)**.

6.2.2 *Aerodynamic stability*

The aerodynamic stability of the bridge should be checked in accordance with DMRB publication BD 49/93 [7].

Bibliography

Standards publications

BS 5400-1:1988, *Steel, concrete and composite bridges — Part 1: General statement.*

BS 5400-2:1978, *Steel, concrete and composite bridges — Part 2: Specification for loads.*

BS 5400-3:2000, *Steel, concrete and composite bridges — Part 3: Code of practice for design of steel bridges.*

BS 5400-4:1990, *Steel, concrete and composite bridges — Part 4: Code of practice for design of concrete bridges.*

BS 5400-5:1979, *Steel, concrete and composite bridges — Part 5: Code of practice for design of composite bridges.*

BS 5400-6:1999, *Steel, concrete and composite bridges — Part 6: Specification of materials and workmanship, steel.*

Other documents

[1] GREAT BRITAIN. *The Building Regulations 1991. Approved Document A (Structure) 1992.* London: The Stationery Office (TSO) (ISBN 0110158873).

[2] GREAT BRITAIN. Department of the Environment, Transport and the Regions. *Design manual for roads and bridges. Volume 1 Highway structures: approval procedures and general design. Section 3: General design: Design of steel bridges: Use of BS 5400-3:1982.* (BD 13) London: The Stationery Office (TSO) (ISBN 0115515275).

[3] GREAT BRITAIN. Department of the Environment, Transport and the Regions. *Manual of contract documents for highway works. Volume 1: Specification for Highway Works.* London: The Stationery Office (TSO). 1998. (ISBN 0115519793).

[4] GREAT BRITAIN. Department of the Environment, Transport and the Regions. *Design manual for roads and bridges. Volume 1 Highway structures: Approval procedures and general design. Section 3: General design. Part 14: Loads for highway bridges.* (BD 37) London: The Stationery Office (TSO) (ISBN 0115514945).

[5] GREAT BRITAIN. Department of the Environment, Transport and the Regions. *Design manual for roads and bridges. Volume 1 Highway structures: Approval procedures and general design. Section 3: General design. Part 1: The design of concrete highway bridges and structures. Use of BS 5400-4:1990.* (BD 24/92) London: The Stationery Office (TSO) (ISBN 0115513043).

[6] GREAT BRITAIN. Department of the Environment, Transport and the Regions. *Design manual for roads and bridges. Volume 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.* (BD 15/92) London: The Stationery Office (TSO) (ISBN 0115513965).

[7] GREAT BRITAIN. Department of the Environment, Transport and the Regions. *Design manual for roads and bridges Volume 1 Highway structures: Approval procedures and general design. Section 3: General design. Part 3: Design rules for aerodynamic effects on bridges.* (BD 49/93). London: The Stationery Office (TSO) (ISBN 0115513558).

ICS 91.010.30; 91.080.10; 91.080.40; 93.040

Descriptors: civil engineering, concrete structure, steel construction, bridges, design, building codes, computation

English version

Eurocode 4: Design of composite steel and concrete structures - Part 2: Composite bridges

**Eurocode 4: Calcul des structures mixtes acier-béton -
Partie 2: Ponts mixtes**

**Eurocode 4: Bemessung und Konstruktion von
Verbundtragwerken aus Stahl und Beton - Teil 2:
Verbundbrücken**

This European Prestandard (ENV) was approved by CEN on 11 August 1997 as a prospective standard for provisional application.

The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into a European Standard.

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

CEN members are the national standards bodies of Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.



EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

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

Contents

Foreword	7
1 General	10
1.1 Scope	10
1.1.2 Scope of ENV 1994-2:1997	10
1.2 Distinction between principles and application rules	10
1.3 Assumptions	10
1.4 Definitions	11
1.4.2 Special terms used in this Part	11
1.4.3 Other definitions	12
1.5 S.I. units	12
1.6 Symbols used in Part 2	12
1.6.1 General	12
1.6.2 Latin upper case letters	13
1.6.3 Greek upper case letters	13
1.6.4 Latin lower case letters	13
1.6.5 Greek lower case letters	14
1.6.6 Subscripts	14
1.6.7 Use of subscripts	15
1.6.8 Conventions for member axes	15
1.7 Normative references	15
2 Basis of design	17
2.2 Definitions and classifications	17
2.2.1 Limit states and design situations	17
2.2.2 Actions	18
2.2.5 Load arrangements and load cases	19
2.3 Design requirements	19
2.3.1 General	19
2.3.2 Ultimate limit states, including fatigue	19
2.3.3 Partial safety factors for ultimate limit states, including fatigue	20
2.3.4 Serviceability limit states	21
2.4 Durability	21
3 Materials	22
3.1 Concrete	22
3.1.1 General	22
3.1.2 Concrete strength classes	22
3.1.3 Shrinkage of concrete	22
3.1.4 Deformability of concrete - elastic theory	23
3.1.5 Deformability of concrete - other theories	23
3.1.6 Thermal expansion	24
3.2 Reinforcing steel	24
3.2.1 General	24
3.2.2 Types of steel	24
3.2.4 Modulus of longitudinal deformation	24

3.3	Structural steel	24
3.3.1	General and scope	24
3.3.2	Yield strength	25
3.3.5	Dimensional and mass tolerances	25
3.3.6	Fracture toughness	25
3.4	Profiled steel sheeting for composite slabs	25
3.5	Connecting devices	25
3.5.2	Shear connectors	25
3.6	Prestressing steel and devices	25
4	Ultimate limit states	26
4.1	Basis	26
4.1.1	General	26
4.2	Properties of cross-sections of beams	27
4.2.1	Effective section	27
4.2.2	Effective width of flanges with respect to shear lag	27
4.2.3	Flexural stiffness	29
4.3	Classification of cross-sections of beams	31
4.3.1	General	31
4.3.2	Classification of steel flanges in compression	31
4.3.3	Classification of steel webs	32
4.4	Resistances of cross-sections of beams	32
4.4.1	Bending moment	32
4.4.2	Vertical shear	35
4.4.3	Bending, axial force, and vertical shear	35
4.4.6	Flange-induced buckling of webs	36
4.5	Global analysis for bridge structures	36
4.5.1	General	36
4.5.3	Elastic analysis	36
4.5.4	Non-linear global analysis	38
4.6	Lateral-torsional buckling of composite beams	38
4.6.1	General	38
4.6.2	Lateral buckling of beams with cross-sections in Class 1 or 2	39
4.6.3	Effects of transverse frames	39
4.7	Tension members in composite bridges	40
4.7.1	General	40
4.7.2	Concrete tension members	40
4.7.3	Composite tension members	41
4.8	Composite compression members	41
4.8.1	Scope	41
4.8.2	General method of design	42
4.8.3	Simplified method of design	42
4.11	Box girders	42
4.12	Fatigue	43
4.12.1	General	43
4.12.2	Fatigue loading and partial safety factors	43
4.12.3	Internal forces	44
4.12.4	Stresses and stress range $\Delta\sigma_E$	45
4.12.5	Fatigue resistance	46
4.12.6	Simplified assessment	47

5	Serviceability limit states	48
5.1	General	48
5.1.1	Scope	48
5.1.2	Classification of structures	48
5.1.3	Global analysis for serviceability limit states	48
5.1.4	Calculation of stresses in cross sections	49
5.2	Limitation of stresses	50
5.3	Crack and decompression control	50
5.3.1	General	50
5.3.2	Minimum reinforcement	51
5.3.3	Control of cracking	54
5.4	Deformations	56
5.5	Vibration	56
6	Shear connection	57
6.1	General	57
6.1.1	Basis of design	57
6.1.2	Deformation capacity of shear connectors	58
6.1.3	Serviceability limit states	58
6.1.4	Ultimate limit states other than fatigue	58
6.1.5	Fatigue assessment based on nominal stress ranges	59
6.1.6	Transient design situations during execution	60
6.2	Longitudinal shear force	60
6.2.1	General	60
6.2.2	Serviceability limit states, and fatigue	61
6.2.3	Ultimate limit states, other than fatigue, for members in Class 1 or 2	61
6.2.4	Local effects of concentrated longitudinal shear force	62
6.2.5	Temperature effects	64
6.2.6	Shrinkage modified by creep	65
6.3	Design resistance of shear connectors	65
6.3.1	General	65
6.3.2	Stud connectors in solid slabs	66
6.3.3	Headed studs used with profiled steel sheeting	67
6.3.5	Hoops in solid slabs	67
6.3.6	Block connectors with hoops in solid slabs	67
6.3.8	Resistance to fatigue of stud connectors in solid slabs	68
6.4	Detailing of the shear connection	69
6.4.1	General recommendations	69
6.4.2	Stud connectors	70
6.4.3	Headed studs used with profiled steel sheeting	70
6.4.5	Hoop connectors	71
6.5	Friction grip bolts	71
6.6	Transverse reinforcement	71
6.6.1	Longitudinal shear in the slab	71
6.6.2	Design resistance to longitudinal shear	72
6.6.3	Contribution of profiled steel sheeting	73
6.6.4	Minimum transverse reinforcement in cast in situ solid slabs	73
6.6.5	Longitudinal splitting	73

7	Composite slabs with profiled steel sheeting, and composite plates	74
7.1	General	74
7.1.1	Scope	74
7.7	Composite plates	74
7.7.1	General	74
7.7.2	Design for local effects	74
7.7.3	Design for global effects	75
7.7.4	Design of shear connectors	75
8	Decks with precast concrete slabs	77
8.1	General	77
8.2	Actions	77
8.3	Partial safety factors for materials	77
8.4	Design, analysis and detailing of the bridge slab	77
8.5	Joints between steel beam and concrete slab	77
8.5.1	Bedding and tolerances	77
8.5.2	Corrosion	78
8.5.3	Shear connection and transverse reinforcement	78
9	Execution	79
9.2	Sequence of construction	79
9.4	Accuracy during construction, and quality control	79
9.4.1	Static deflection during and after concreting	79
9.4.3	Shear connection	79
9.4.4	Composite slabs with profiled steel sheeting	80
10	Design assisted by testing	81
10.1	General	81
10.3	Testing of composite floor slabs	81

Annex A Reference documents (not applicable)

Annex K Filler beam decks (Normative)	82
K.1 General	82
K.2 Requirements	83
K.3 Global analysis	83
K.4 Ultimate limit states	85
K.4.1 General	85
K.4.2 Bending moments	85
K.4.3 Vertical shear	85
K.4.4 Strength and stability of steel beams during construction	86
K.5 Serviceability limit states	86
K.5.1 General	86
K.5.2 Cracking of concrete	86
K.5.3 Minimum reinforcement	86
K.5.4 Control of cracking	86
K.6 Detailing	86
K.7 Half-through bridges with transverse filler beams	87
K.7.1 General	87
K.7.2 Analysis	87
K.7.3 Shear in the direction of span of the transverse beams	87

K.7.4	Detailing	88
Annex L	Effects of tension stiffening in composite bridges (Informative)	89
L.1	Scope	89
L.2	Tension members in bowstring arches and trusses	89
L.3	Tension members in composite beams	91
L.4	Stiffness	96
L.5	Calculation of the stress range in reinforcing, prestressing and structural steel for fatigue loading	96
L.5.1	General	96
L.5.2	Stress ranges in reinforcing and prestressing steel	96
L.5.3	Stress ranges in structural steel	98
L.5.4	Range of longitudinal shear per unit length, $\Delta v_{f,E}$, for shear connectors	98

Foreword

rep.

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

Background to the Eurocode Programme

- (4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonised technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the 'Structural Eurocodes'.
- (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 actions on structures
EN 1992	Eurocode 2	Design of concrete structures
EN 1993	Eurocode 3	Design of steel structures
EN 1994	Eurocode 4	Design of composite steel and concrete structures
EN 1995	Eurocode 5	Design of timber structures
EN 1996	Eurocode 6	Design of masonry structures
EN 1997	Eurocode 7	Geotechnical design
EN 1998	Eurocode 8	Design of structures for earthquake resistance
EN 1999	Eurocode 9	Design of aluminium alloy structures

- (8) A separate sub-committee has been formed by CEN/TC250 for each of the Eurocodes listed above.
- (9) This Part of ENV 1994 is being published as a European Prestandard (ENV) with an initial life of three years.

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

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

(12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of Sub-committee CEN/TC250/SC4 at the following address :

National Standards Authority of Ireland, Glasvenin, Dublin 9, Ireland
Telephone international 353 1 807 38 00
Fax international 353 1 807 38 38

or to your national Standards Organisation.

National Application Documents

(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 review the "boxed values" and may substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the necessary 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 (NAD) giving any mandatory values to be substituted for "boxed" values, referencing compatible supporting Standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organisation.

(15) It is intended that this Prestandard will be used in conjunction with the particular NAD valid in the country in which the bridge is to be located.

Matters specific to this prestandard

(16) The scope of Eurocode 4 is defined in clause 1.1 of ENV 1994-1-1:1992 and the scope of this part of Eurocode 4 is defined in 1.1.2.

(17) Bridges are essentially public works, for which :

- the European Directive 93-37/CEC on Public Procurement is particularly relevant, and
- public authorities have responsibilities as owners.

Within this context this Prestandard has been established with two main objectives :

- to be sufficiently precise and comprehensive for contractual use,
- to be sufficiently flexible to allow the intervening parties fully to exert their technical responsibilities.

(18) Because of the responsibilities of relevant authorities for bridges, it has been anticipated that, for the application of this Part, it will be supplemented by :

- general complementary rules and options to be provided by National Application Documents (NAD - see (14)) and
- complementary and/or modifying specifications for particular projects.

Wherever this Prestandard mentions "unless otherwise specified", it is intended that the relevant authorities (to be identified, if relevant, in the particular NADs) will remain free to intervene at each of these two levels. It is the same where this Prestandard refers to the "client", if the client is not the relevant authority itself.

(19) Concerning the treatment of γ_M for structural steel, 0.5.2 of ENV 1994-1-1:1992 is applicable.

(20) The framework and structure of this Part 2 correspond to ENV 1994-1-1:1992. However, ENV 1994-2:1997 contains Principles and Application Rules which are specific to composite bridges.

(21) Some Principles and Application Rules in ENV 1994-1-1:1992 are modified or replaced. The new provisions are identified by the symbols *mod.* or *rep.*, respectively.

(22) Where a new Principle or Application Rule is added, it is identified by the symbol *add.*

(23) Any clause, subclause, or paragraph of ENV 1994-1-1:1992 that is neither modified or replaced, is applicable. These provisions are not repeated in this ENV 1994-2:1997.

(24) For the application of this Part 2 it is assumed that the intervening parties :

- select from ENV 1991-3:1995 the relevant traffic load models and take the necessary complementary decisions about actions,
- define, with regard to the type of bridge under consideration (see 1.1.2) and the environmental conditions of exposure, the verification criteria for the serviceability limit states.

1. General

1.1 Scope

1.1.2. Scope of ENV 1994-2:1997

rep.

- (1) P ENV 1994-2:1997 gives a general basis for the design of composite bridges.
- (2) P In addition ENV 1994-2:1997 gives a specific basis for the design of composite structures and members for bridges such as road, railway, and pedestrian bridges, and detailed rules for composite bridge structures such as beam and slab bridge decks, box girders, trusses and columns that support bridge decks.
- (3) No application rules are given for the use of unbonded tendons or for cable stayed bridges.
- (4) P For the use of composite slabs in bridges, see 7.1.1.
- (5) P For the use of filler beam decks, see annex K.
- (6) P Provisions for the design of high strength cables, bearings and expansion joints are given in annexes A, B and E of ENV 1993-2:1997.
- (7) P The implicit inclusion of a type of bridge or a form of structure (as defined in 1.4.1(2)) does not imply that all details of its design are covered comprehensively.
- (8) For the execution of steel structures, reference should be made to ENV 1090-5.

1.1.3 Further Parts of ENV 1994

Clause 1.1.3 does not apply.

1.2 Distinction between principles and application rules

- (3) P The principles are identified by the letter P following the paragraph number.
mod.
- (6) In this Part, Application Rules have only a paragraph number, e.g. as this paragraph.
mod.

NOTE : Tables and figures have the same status as the paragraphs to which they relate.

1.3 Assumptions

rep.

- (1) P The assumptions given in clause 1.3(1) of ENV 1992-1-1:1991 and 1993-1-1:1992 are applicable.
- (2) P The design procedures are valid only when the requirements for execution and workmanship given in Section 9 are also complied with.
- (3) P For numerical values identified by □, see Foreword paragraph (13).

1.4 Definitions

1.4.2 Special terms used in this Part

rep.

1.4.2.1 Frame : A structure or portion of a structure, comprising an assembly of directly connected structural members, designed to act together to resist load. This term covers both plane frames and three-dimensional frames.

1.4.2.2 Filler beam deck : see K.1.(1) P

1.4.2.3 Composite member : A structural member with components of concrete and of structural or cold-formed steel, interconnected by shear connection so as to limit the longitudinal slip between concrete and steel and the separation of one component from the other.

1.4.2.4 Composite bridge : A bridge in which at least some of the principal members are composite members.

1.4.2.5 Propped structure or member : A structure or member where the weight of concrete elements is applied to the steel elements which are supported within the span, or is carried independently until the concrete elements are able to resist stresses.

1.4.2.6 Unpropped structure or member : A structure or member in which the weight of concrete elements is applied to the steel elements which are unsupported within the span.

1.4.2.7 Shear connection : An interconnection between the concrete and steel components of a composite member that has sufficient strength and stiffness to enable the two components to be designed as parts of a single structural member.

NOTE : The concept of partial shear connection as used in ENV 1994-1-1:1992 is not applicable to bridges.

Except as provided in 4.8.2 and annex K shear connection means mechanical shear connection that does not rely on bond or adhesion at interfaces between steel and concrete.

1.4.2.8 Composite column : A composite member subjected mainly to compression and bending. Only columns with cross-sections of the types defined in 4.8.1 are treated in this Eurocode.

1.4.2.9 Composite beam : A composite member subjected mainly to bending.

1.4.2.10 Continuous composite beam : A beam with three or more supports, in which the steel section is either continuous over internal supports or is jointed by full-strength and rigid connections, with connections between the beam and each support such that it can be assumed that the support does not transfer significant bending moment to the beam.

1.4.2.11 Global analysis : The determination of a consistent set of internal forces and moments in a structure which are in equilibrium with a particular defined set of actions on the structure, and are based on the properties of the materials.

1.4.2.12 Clearance gauge : The maximum height authorised for vehicles running under a bridge.

1.4.2.13 Composite plate : Composite member subjected mainly to bending, consisting of a flat plate connected to a concrete slab, in which both the length and width are much greater than the thickness.

1.4.3 Other definitions

add.

- (1) P The definitions of clause 1.4 of ENV 1991-3:1995 apply.
- (2) P The definitions of clause 1.4 of ENV 1992-2:1996 apply.
- (3) P The definitions of clause 1.4 of ENV 1993-2:1997 apply.
- (4) P For the verifications relating to fatigue, the definitions given in 9.1.5 of ENV 1993-1-1:1992 apply.
- (5) P The definitions of clause 1.4 of ENV 1993-1-5:1997 apply.
- (6) For isostatic effects and hyperstatic effects of shrinkage and differential temperature, see 2.2.2.1(4) of ENV 1994-1-1 : 1992.

1.5 S.I. Units

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

mod.

- Forces and loads : kN or MN units
- unit mass : kg/m³
- unit weight : kN/m³
- stresses and strengths : N/mm² (= MN/m² or MPa)
- moments (bending) : kNm or MNm

1.6 Symbols used in Part 2

rep.

1.6.1 General

- (1) Only the main symbols are defined in this Section. Symbols which are used only in small parts of this Eurocode are defined where they appear.

NOTE : The following list of symbols include the principal combinations of symbols and subscripts in this Eurocode. The list does not include symbols used in one place only, nor those symbols used in ENV 1992-2:1996, ENV 1993-2:1997 and ENV 1993-1-5:1997 but not directly in this Part.

NOTE : The notation used is based on ISO 3898:1987.

1.6.2 Latin upper case letters

- A* Accidental action; Area
- C* Fixed value; Factor
- E* Effect of actions; Modulus of elasticity
- F* Action; force
- G* Permanent action; shear modulus
- I* Second moment of area
- K* Stiffness factor (*I/L*)

L	Length; Span; System length
M	Moment in general; Bending moment
M_{Rd}	Design value of the resisting bending moment
M_{sd}	Design value of the applied internal bending moment
N	Axial force; Number of shear connectors, Number of cycles
P	Prestressing
P_R	Shear resistance of a shear connector
Q	Variable action
R	Resistance
S	Internal forces and moments (with subscripts d or k)
V	Shear force
W	Section modulus
X	Value of a property of a material

1.6.3 Greek upper case letters

Δ	Difference in (precedes main symbol)
----------	--

1.6.4 Latin lower case letters

a	Constant; Distance; Geometrical data; Constant
b	Width; Breadth
c	Distance; Outstand; Thickness of concrete cover
d	Diameter; Depth
e	Eccentricity
f	Strength (of a material)
f_{ck}	Characteristic compressive strength of concrete
f_{sk}	Characteristic tensile yield strength of reinforcement
f_u	Specified ultimate tensile strength of the material of a stud, a bolt, a rivet ...
f_y	Nominal tensile yield strength of structural steel
f_{yp}	Characteristic (nominal) tensile yield strength of profiled steel sheeting
h	Height
i	Radius of gyration
k	Coefficient; Factor
l	(or ℓ or L) Length; Span; Buckling length (Note : l can be replaced by L or by ℓ (handwritten) for certain lengths or to avoid confusion with 1 (numeral).)
m	Factor for composite slabs; Slope constant of fatigue strength curve
n	Modular ratio
r	Radius
s	Spacing; Distance
v	Longitudinal shear force per unit length
w	Crack width
xx, yy, zz	Rectangular axes

1.6.5 Greek lower case letters

α	Angle; Ratio; Coefficient of linear thermal expansion; Factor
β	Angle; Ratio; Factor
γ	Partial safety factor (always with appropriate subscript : e.g., F, G, Q, A, M, Ma, a, ap, c, s, v, Rd)
δ	Steel contribution ratio; Deflection
ε	Strain; Coefficient

η	Coefficient
θ	Angle; Slope
λ	(or $\bar{\lambda}$) Factor for range of stress; Slenderness ratio
μ	Coefficient of friction; Moment ratio
ν	Poisson's ratio
ρ	Unit mass; Reinforcement ratio
τ	Shear stress
φ	Diameter of a reinforcing bar
χ	Reduction factor (for buckling)
ψ	Factors defining representative values of variable actions; Stress ratio
ξ	Ratio of bonding strength

1.6.6 Subscripts

A	Accidental
a	Structural steel
b	Buckling; Bolt; Beam; Bottom
c	Compression; Concrete; Composite cross section
cr	(or crit) Critical; Cracking
cs	Concrete shrinkage
d	Design
dst	Destabilising
eff	Effective
E	Equivalent
e	Effective (with further subscript)
el	Elastic
f	Flange ; Fatigue
F	Fatigue
G	Permanent (referring to actions)
h	Haunch; Horizontal
i	Index (replacing a numeral)
inf	Inferior; Lower
k	Characteristic
l	(or ℓ) Longitudinal
LT	Lateral - torsion
M	Material
m	Allowing for axial force; Mean
nom	Nominal
P	Prestressing
p	(possibly supplementing a) Profiled steel sheeting
pl	Plastic
Q	Variable (referring to actions)
R	Resistance
r	Reduced
S	Internal force; Internal moment
s	Reinforcing steel
se	Neglecting concrete in stress calculation
st	Steel
stb	Stabilising
sup	Superior; Upper
t	Tension; Tensile, Transversal; Top
ten	Tension

ts	Tension stiffening
u	Ultimate
v	Vertical; Related to shear connection
w	Web
x	Axis along member
y	Major axis of cross-section
z	Level arm
0,1,2, etc...	Particular values

1.6.7 Use of subscripts

- (1) Reference should be made to 1.6.6 of ENV 1993-1-1:1992.

1.6.8 Conventions for member axes

- (1) Reference should be made, if relevant, to 1.6.7 of ENV 1993-1-1:1992.

1.7 Normative references

add.

- (1) This European Prestandard incorporates, by dated and undated references, provisions from other publications. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Prestandard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

- (2) When using ENV 1994-2:1997, reference should be made to the following ENVs, where relevant:

ENV 1090-1	Execution of steel structures Part 1: General rules and rules for buildings
ENV 1090-5	Execution of steel structures Part 5: Supplementary rules for bridges and plated structures.
ENV 1991-1	Eurocode 1: Basis of design and actions on structures Part 1: Basis of design
ENV 1991-2-1	Eurocode 1: Basis of design and actions on structures Part 2.1: Densities, self-weight and imposed loads
ENV 1991-2-4	Eurocode 1: Basis of design and actions on structures Part 2.4: Wind loads
ENV 1991-2-5	Eurocode 1: Basis of design and actions on structures Part 2.5: Thermal actions
ENV 1991-2-6	Eurocode 1: Basis of design and actions on structures Part 2.6: Loads and deformations imposed during execution
ENV 1991-2-7	Eurocode 1: Basis of design and actions on structures Part 2.7: Accidental actions
ENV 1991-3	Eurocode 1: Basis of design and actions on structures Part 3: Traffic loads on bridges
ENV 1992-1-1	Eurocode 2: Design of concrete structures Part 1.1 : General rules and rules for buildings
ENV 1992-1-3	Eurocode 2: Design of concrete structures Part 1.3 :General rules - Precast concrete elements and structures
ENV 1992-1-4	Eurocode 2: Design of concrete structures

	Part 1.4 :General rules - Lightweight aggregate concrete with closed structure
ENV 1992-1-5	Eurocode 2: Design of concrete structures Part 1.5 :General rules - Structures with unbonded and external prestressing tendons
ENV 1992-1-6	Eurocode 2: Design of concrete structures Part 1.6 :General rules - Plain concrete structures
ENV 1992-2	Eurocode 2: Design of concrete structures Part 2: Concrete bridges
ENV 1992-3	Eurocode 2: Design of concrete structures Part 3: Concrete foundations
ENV 1993-1-1	Eurocode 3: Design of steel structures Part 1.1: General rules and rules for buildings
ENV 1993-1-3	Eurocode 3: Design of steel structures Part 1.3: Supplementary rules for cold formed thin gauge members and sheeting
ENV 1993-1-5	Eurocode 3: Design of steel structures Part 1.5: Supplementary rules for the strength and stability of planar plated structures
ENV 1993-2	Eurocode 3: Design of steel structures Part 2: Steel bridges
ENV 1994-1-1	Eurocode 4: Design of composite steel and concrete structures Part 1.1: General rules and rules for buildings
ENV 1997-1	Eurocode 7: Geotechnical design Part 1: General rules
ENV 1998-2	Eurocode 8: Earthquake resistant design of structures Part 2: Bridges

(3) P The reference standards are those in ENV 1991-1:1994, ENV 1992-2:1996 and ENV 1993-2:1997. Stud welding shall be in accordance with prEN ISO 14555 'Welding - Arc stud welding of metallic materials' and prEN ISO 13918 'Welding-Studs for arc stud welding'.

2. Basis of design

2.2 Definitions and classifications

2.2.1 Limit states and design situations

2.2.1.1 Limit states

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

mod.

- 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 shear connection, supports and foundations,
- failure caused by fatigue.

Limit states may also concern only concrete or steel parts of the structure (e.g. the steel part during an erection phase), for which reference should be made to ENV 1992-2 : 1996, ENV 1993-1-5 : 1997 and ENV 1993-2 : 1997 as appropriate.

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

mod.

- deformations or deflections which adversely affect the appearance or effective use of the structure (including the proper functioning of services) or cause damage to finishes or non-structural elements,
- vibration which causes discomfort to people, damage to the structure or its contents, or which limits its functional effectiveness,
- cracking of the concrete which is likely to affect appearance, durability or water-tightness adversely,
- damage to concrete because of excessive compression, which is likely to lead to loss of durability,
- slip at the steel-concrete interface when it becomes large enough to invalidate design checks for other serviceability limit states where the effects of slip are neglected,
- excessive creep and microcracking of concrete, and irreversible behaviour of the structure, caused by excessive stresses.

2.2.1.2 Design situations

(2) Attention is drawn to the necessity of identifying and considering, when relevant, *mod* several transient situations corresponding to the method of construction and to the successive phases in the sequence of the bridge erection.

2.2.2 Actions

2.2.2.2 Characteristic values of actions

(7)P During execution loads shall be calculated according to ENV 1991-2-6 : 1997.
add.

2.2.2.3 Representative values of variable actions

(2)P Other representative values are related to the characteristic value Q_k by means of a *mod.* factor ψ_i . These values are defined as:

- combination value : $\psi_0 Q_k$ (see 2.3.2.2 and 2.3.4)
- infrequent value : $\psi'_1 Q_k$ (see 2.3.2.2 and 2.3.4)
- frequent value : $\psi_1 Q_k$ (see 2.3.2.2 and 2.3.4)
- quasi-permanent value : $\psi_2 Q_k$ (see 2.3.2.2 and 2.3.4)

(4)P Factors ψ_i applicable to some relevant actions are given in ENV 1991-1-3 : 1995 and *mod.* in ENV 1991-2-6 : 1997. Values of such factors ψ_i for any action not given should be selected with due regard to the physical characteristics of the action.

2.2.2.4 Design values of actions

(2)P Specific examples of the use of γ_F are:
mod.

$$G_d = \gamma_G G_k$$

$$Q_d = \gamma_Q Q_k \text{ or } \gamma_Q \psi_i Q_k$$

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

$$P_d = \gamma_p P_k$$

2.2.5 Load arrangements and load cases

NOTE: detailed rules on load arrangements and load cases are given in ENV 1991-1 : 1994.

(1)P A load arrangement identifies the position, magnitude and direction of a free action.
mod. Traffic loads shall be in accordance with ENV 1991-3 : 1995.

(4) and (5) do not apply.

2.3 Design requirements

2.3.1 General

(5)P Predicted settlements shall be taken into account when they have a significant effect
add. on the structural behaviour.

(6) Where settlements are taken into account, appropriate estimated values of predicted
add. settlements should be used.

2.3.2 Ultimate limit states, including fatigue

2.3.2.1 Verification conditions

(3) does not apply.

(5)P When considering a limit state of failure induced by fatigue, see 4.12, 6.1.5, 6.3.8 and
add. 7.7.4.

2.3.2.2 Combinations of actions

rep.

(1)P For road bridges, foot bridges and railway bridges, the combinations of actions are defined in ENV 1991-3 : 1995. For other categories of bridges they shall be specified in the project specifications or by the relevant authority.

2.3.2.3 Design values of permanent actions

(3)P Where a single permanent action is treated as consisting of separate unfavourable
mod. and favourable parts, allowance may be made for the relationship between these parts by adopting special design values (see 2.3.3.1).

(5) For continuous beams and frames, the same values of the partial safety factors
mod. for self-weight of the structure (evaluated as in 2.3.2.3 (3)) may be applied to all spans, except for cases involving the static equilibrium of cantilevers or uplift at bearings.

2.3.2.4 Verification of static equilibrium

mod.

(1) For verification of static equilibrium see 2.3.1 and 5.1.7 of ENV 1993-2 : 1997.

2.3.3 Partial safety factors for ultimate limit states, including fatigue *rep.*

2.3.3.1 Partial safety factors for actions on bridge structures

- (1)P Partial safety factors shall be taken from the appropriate part of ENV 1991.
- (2) For verification of fatigue see 4.12.2.
- (3) Where, according to 2.3.2.3 (3)P, favourable and unfavourable parts of a permanent action need to be considered as individual actions, see C. 2.3(2) of ENV 1991-3 : 1995.
- (4) For prestress by bonded tendons see ENV 1992-2 : 1996.
- (5) For prestress imposed by jacking at supports the partial safety factor $\gamma_P = \boxed{1,0}$ should be used for ultimate limit states.
- (6)P The partial safety factor to be assumed for the nominal free shrinkage strain at ultimate limit states shall be $\boxed{1,0}$.

2.3.3.2 Partial safety factors for resistances and material properties

- (1)P Except in certain cases mentioned in 2.2.3.2 (2)P and (3) the factors γ_M shall be applied to lower characteristic or nominal strengths of materials (following 2.2.3.2 (1)), and are as given in Table 2.3.

Table 2.3: Partial safety factors for resistances and material properties

Combination	Structural steel		Concrete	Steel reinforcement and Prestressing tendons	Profiled steel sheeting
	γ_a	γ_{Rd}			
Fundamental	$\boxed{1,0}$	$\boxed{1,10}$	$\boxed{1,5}$	$\boxed{1,15}$	$\boxed{1,10}$
Accidental (except earthquakes)	$\boxed{1,0}$	$\boxed{1,0}$	$\boxed{1,3}$	$\boxed{1,0}$	$\boxed{1,0}$

(2)P The values in Table 2.3 are assumed to take account of, inter alia, differences between the strength of test specimens of the structural materials and their strength in situ. They are applicable to some elastic mechanical properties, but only in cases specified in the relevant clauses; in other cases they should be substituted by $\gamma_M = 1,0$. For physical non-mechanical coefficients (e.g. density, thermal expansion), γ_M shall be taken as equal to 1,0.

(3) Where values of γ_c lower than given in table 2.3 are used for precast elements, they should be justified by adequate quality assurance procedures (see 2.3.3.2(4) of ENV 1992-1-1 : 1991). This applies only where the precast concrete element constitutes the full structural depth of the slab.

(4)P Values of γ_M for shear connection are given in 6.3.2.1 (as γ_v) for studs, 6.3.4 of ENV 1994-1-1 : 1992 for block connectors, 6.3.5 for hoops and 6.3.7 of ENV 1994-1-1 : 1992 for angle connectors.

(5)P Values of γ_M for bolts, rivets, pins, welds, and slip resistance of bolted joints are as given in 6.1(2) of ENV 1993-2 : 1997.

(6)P Where structural properties are determined by testing, reference shall be made to Annex D of ENV 1991-1 : 1994

2.3.4 Serviceability limit states

(2)P The combinations of actions for serviceability limit states are defined in 9.5.2 of *rep.* ENV 1991-1 : 1994 and C.3.2 of ENV 1991-3 : 1995. Imposed deformations shall be introduced as best estimate (mean) values.

(4) and (5) do not apply.

(6) Values of γ_M shall be taken as $\boxed{1,0}$, except where stated otherwise in particular *clauses.*
mod. For structural steel this γ_M corresponds to $\gamma_{M,ser.}$ given in ENV 1993-2 : 1997.

(7) For prestress by bonded tendons see ENV 1992-1-1 : 1991.
add.

2.4 Durability

(3) Clause 4.1 of ENV 1992-2 : 1996 is applicable.
rep.

(4) Clause 2.2.5 of ENV 1993-2 : 1997 is applicable.
add.

3. Materials

NOTE: This Section is concerned with the properties of materials that are particularly relevant to composite bridges.

3.1 CONCRETE

3.1.1 General

mod.

(1) Normal-weight aggregate concrete strength classes higher than C50/60 and light-weight aggregate concrete strength classes higher than LC50/60 should not be used unless their use is appropriately justified. No Application Rules are given for these cases. Density classes 1,6 or above should be used.

(2) Strength classes lower than C30/37, LC 30/37 should not be used.

3.1.2 Concrete strength classes

mod.

(1)P This Eurocode is based on the characteristic cylinder strength, f_{ck} , measured at age 28 days in accordance with 3.1.2.2 of ENV 1992-1-1 : 1991.

(2) The design should be based on a strength class of concrete which corresponds to a specified value of f_{ck} .

(3) For normal-weight aggregate concrete, 3.1.2.4 of ENV 1992-1-1 : 1991 gives for the different strength classes the characteristic strength f_{ck} and the corresponding values of the associated cube strength (e.g. the classification of concrete C 30/37 refers to cylinder/cube strengths) and of the mean tensile strength f_{ctm} and characteristic tensile strengths $f_{ctk 0.05}$ and $f_{ctk 0.95}$.

(4) For lightweight aggregate concrete, 3.1.2.4 of ENV 1992-1-4 : 1994 gives for the different strength classes the characteristic strength f_{ck} and the corresponding values of the associated cube strength (e.g. the classification of concrete LC 30/37 refers to cylinder /cube strengths).

(5) For lightweight aggregate concretes, tensile strengths are given by 3.1.2.3 of ENV 1992-1-4 : 1994.

3.1.3 Shrinkage of concrete

mod.

(1) To provide accurate control of the profile during execution and particularly where the free shrinkage is expected to take exceptional values because of the composition of the concrete, of its environment (e.g. very frequently wet concrete), of its thickness, of the quality of the aggregates, of the percentage of reinforcement, or when shrinkage has to be assessed at intermediate times, reference should be made to 3.1.2.5.5 and Appendix 1 of ENV 1992-1-1 : 1991 for the evaluation of the effects due to the drying shrinkage.

(2) In the most common cases generally and unless specified for the particular project, the free shrinkage strain values from setting of the concrete, ϵ_{cs} , may be taken from 3.1.2.5.5 of ENV 1992-1-1 : 1991, with values of consistence classes of concretes S2 and S3 (see 7.2.1 in ENV 206 : 1990).

(3) In addition to the verifications considered in 3.1.3(1), for slabs which are connected to the steel from the time of casting, where the structure is in categories A, B, or C according to 4.4.0.3 of ENV 1992-2 : 1996, and is prestressed, the following effects should be considered for the control of cracking and deformations during execution, where relevant:

- the autogenous shrinkage, caused by the volume reduction of the cement paste;
- the thermal shrinkage caused by the heat of hydration of the cement.

3.1.4. Deformability of concrete - elastic theory

3.1.4.1 Secant modulus of elasticity for short-term loading *mod.*

(1) Nominal values of the mean secant modulus E_{cm} for short-term loading for normal-weight concrete of a given strength class or of characteristic compressive strength f_{ck} are given in 3.1.2.5.2 of ENV 1992-1-1 : 1991.

(2) does not apply.

(3) For lightweight concretes, secant moduli are given in 3.1.2.5.2 of ENV 1992-1-4 : 1994.

3.1.4.2 Creep *mod.*

(1) Allowance should be made for the deformation of concrete due to creep in accordance with 2.5.5.1 (1 to 10 and 13), of ENV 1992-1-1 : 1991 and 3.1.2.5.5 of ENV 1992-1-4 : 1994, where it is relevant in the ultimate limit state or in the serviceability limit state.

3.1.5 Deformability of concrete - other theories *mod.*

(1) Where a non-linear global analysis (4.4.1.4), or a non-linear section analysis (4.4.1.3) is used, reference should be made to 4.2.1.3.3 of ENV 1992-1-1 : 1991.

3.1.6 Thermal expansion *mod.*

(1) The nominal value of the coefficient of linear thermal expansion α_T may be taken as indicated in ENV 1991-2-5 : 1996.

(2) The difference between the coefficients of thermal expansion of structural steel and of concrete may normally be neglected in design for normal-weight aggregate.

3.2 Reinforcing steel

3.2.1 General

mod.

- (1) Clause 3.2 of ENV 1992-1-1 : 1991 is applicable to composite structures.

3.2.2 Types of steel

- (1)P The steels covered by ENV 1994-2 shall be distinguished as follows:

mod.

– according to surface characteristics:

(a) plain smooth bars or wires and

(b) ribbed bars or wires (including welded fabric), resulting in high bond action (as specified in ENV10080 : 1995).

- according to ductility characteristics: high or normal, as defined in 3.2.4.2 (2) of ENV 1992-1-1 : 1991.

- according to weldability: 3.2.5.2 and 4.2.2.4.2 of ENV 1992-1-1 : 1991 are applicable.

3.2.4 Modulus of longitudinal deformation

mod.

- (1) The value of the modulus of longitudinal deformation E_s may be assumed as 200 kN/mm² in accordance with ENV 1992-1-1 : 1991. However, for simplicity, the same value may be assumed as for structural steel in accordance with ENV 1993-1-1 : 1992, i.e. $E_s=210$ kN/mm².

3.3 Structural steel

3.3.1 General and scope

- (1)P ENV 1994-2 covers the design of composite bridges fabricated from steel materials
mod. conforming to Section 3 of ENV 1993-2 : 1997.

- (2)P Clause 3.2 of ENV 1993-2 : 1997 is applicable to composite structures.
mod.

- (3) does not apply.

3.3.2 Yield strength

mod.

- (1) The nominal values of the yield strength f_y and the ultimate tensile strength f_u for hot rolled steel should be taken from 3.2.2(1)P of ENV 1993-2 : 1997.
- (2) These nominal values may be adopted as characteristic values in calculations.

3.3.5 Dimensional and mass tolerances

mod.

- (1)P Clause 3.2.6 of ENV 1993-2 : 1997 shall be applied.

3.3.6 Fracture toughness

add.

- (1)P Clause 3.2.4 of ENV 1993-2 : 1997 shall be applied.

3.4 Profiled steel sheeting for composite slabs

- (1) Unless specified otherwise in the project specifications or by the relevant authority, *rep.* clause 3.4 of ENV 1994-1-1 : 1992 is applicable. See also 7.1.1(5).

3.5 Connecting devices

3.5.2 Shear connectors

- (1), (2) and (3) do not apply. See Clause 6.3.

(4)P The material of the connector shall be of a quality which takes into account *mod.* its required performance and the method of fixing to the structural steelwork and, for welded connectors, the welding technique to be used. Where hoops act as shear connectors, special care shall be taken that the material is of an appropriate weldable quality.

- (7) *mod.* The dimensions of stud connectors, including the weld collar, should be in accordance with prEN ISO 13918 'Welding - Studs for arc stud welding'.

3.6 Prestressing steel and devices

add.

- (1)P The relevant clauses of ENV 1992-1-1 : 1991 shall be applied.

4 Ultimate limit states

4.1 Basis

4.1.1 General

(1)P The scope of this Section is composite bridges and their components, except that *mod.* design of shear connection in beams and in-plane shear in a concrete flange are treated in Section 6. Filler beam decks are treated in annex K and composite plates in Section 7.

(2)P Composite bridges and their components shall be so proportioned that the basic *mod* design requirements for the ultimate limit state given in Section 2 are satisfied. The relevant design requirements given in Sections 2 of ENV 1992-2:1996 and ENV 1993-2:1997 shall also be satisfied.

(3)P The requirements of 2.3.1 of ENV 1993-2:1997 concerning static equilibrium shall *mod.* be satisfied.

(4)P In analyses of composite bridges, their components, and cross-sections, appropriate *mod.* account shall be taken of the properties of concrete, reinforcing steel, prestressing steel and structural steel defined in Section 3 and the properties of shear connectors defined in Sections 3 and 6. Account shall be taken of loss of resistance or ductility associated with buckling of steel, and with cracking, crushing, or spalling of concrete.

(6) and (7) do not apply.

(8) The effects of creep of concrete on both for global and cross-section analyses for *mod.* bridges may be allowed for by the use of modular ratios.

(9) does not apply.

(10) Where a deck slab spans longitudinally between composite cross beams, and is *add.* subjected to global longitudinal force, there may also be local effects of the actions that cause the global force. Possible combination of these effects should be considered, where the width of the local actions is comparable with the effective width. This applies to verifications of the ultimate limit states other than fatigue, and may apply to fatigue verifications for decks of composite bridges in category D or E according to 4.4.0.3 of ENV 1992-2:1996.

4.1.2 Beams (does not apply)

4.1.3 Composite columns, frames and connections (does not apply)

4.2 Properties of cross-sections of beams

4.2.1 Effective section

- (1)P Allowance shall be made for the flexibility of concrete or steel flanges in in-plane shear (shear lag) either by means of rigorous analysis, or by using an effective width of flange in accordance with 4.2.2.
- (2) does not apply.
- (3) Where plastic analysis of cross sections is used, for bars in tension only reinforcement of high ductility, as defined in 3.2.4.2 of ENV 1992-1-1:1991, should be included in the effective section.
- (5) does not apply.
- (6)P The effective cross-section properties of structural steel compression elements in Class 4, as defined in 4.3.1, shall be based on the gross areas of the steel sections in global analysis and on effective areas in accordance with ENV 1993-1-5:1997, for the analysis of sections.

4.2.2 Effective width of flanges with respect to shear lag *rep.*

4.2.2.1 Effective width for global analysis

- (1) For composite flanges, including composite plates according to Section 7, a constant effective width may be assumed over the whole of each span. This value may be taken as the value at midspan according to 4.2.2.2.
- (2) For steel flanges ENV 1993-1-5:1997 is applicable.

4.2.2.2 Effective width for verification of cross-sections

- (1) For steel flanges ENV 1993-1-5:1997 is applicable.
- (2) For verifications for the ultimate limit state and the limit state of fatigue for composite flanges, including composite plates according to Section 7, the effective width may be determined from equation (4.1), where b_{eff} is shown in figure 4.1 for a typical cross section:

$$b_{\text{eff}} = b_0 + \sum b_{\text{ei}} \quad (4.1)$$

where

b_0 is the distance between the centres of the outstand shear connectors according to figure 4.1. For composite plates 7.7.1(3) should be considered,

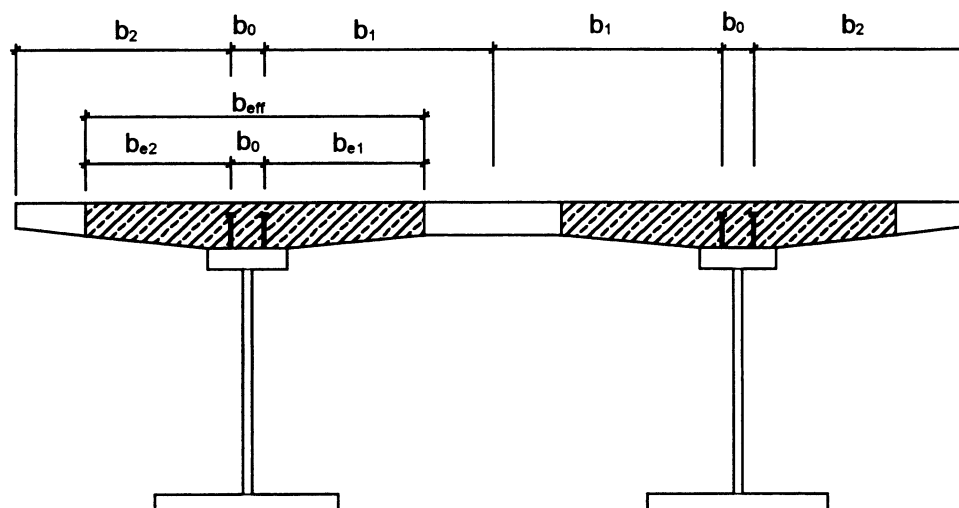


Figure 4.1: Definition of effective width

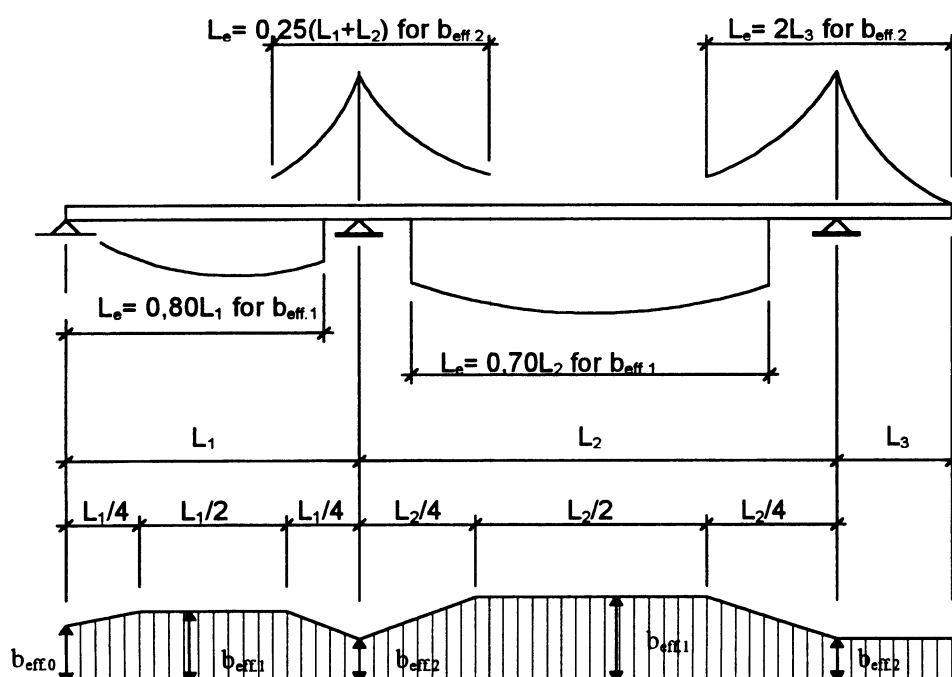


Figure 4.3: Determination of the equivalent spans L_e for the effective width of concrete flanges and distribution of the effective width over the span length

b_{ei} is the value of the effective width of the concrete flange on each side of the web. It should be taken as $L_e / 8$ but not greater than the geometric width b . The length L_e is the approximate distance between points of zero bending moments according to (3) to (5). The distribution of the effective width between internal supports and midspan regions may be assumed to be as shown in figure 4.3.

The effective width $b_{\text{eff},0}$ at end supports should be taken as

$$b_{\text{eff},0} = b_0 + \sum \beta_i \cdot b_{ei} \quad \text{with } \beta_i = (0,55 + 0,025 L_e / b_i) \leq 1,0 \quad (4.2)$$

where b_{ei} is the effective width of the end span at midspan and L_e is the equivalent span of the end span according to figure 4.3.

- (3) In cases where the design is governed by a moment envelope from various load arrangements (e.g. continuous beams under traffic loads) for continuous beams and cantilevers L_e may be assumed to be as shown in figure 4.3.
- (4) In a verification for a transient design situation (e.g. permanent loads and imposed deformations by jacking of supports during erection) the bending moment distribution may be subdivided at the contraflexure points into individual lengths of L_e .
- (5) For cross-sections with bending moments resulting from the main-girder system and from a local system (for example, a truss bridge with direct actions on the chord between nodes) the relevant effective widths for the main-girder system and the local system should be used for the relevant bending moments.
- (6) The transverse distribution of stresses due to shear lag may be taken in accordance with clause 3.3 of ENV 1993-1-5:1997.

4.2.2.3 Effective width for local load introduction

- (1) Unless a more accurate method is used, for the dispersion area of concentrated forces in the longitudinal direction for concrete elements 4.5.3.2.7 of ENV 1992-1-1:1991 is applicable.
- (2) The local load-introduction of in-plane forces of steel elements should be verified in accordance with ENV 1993-1-5:1997.

4.2.3 Flexural stiffness

- (1) Unless a more precise method is used the elastic section properties of a composite *mod.* cross-section with concrete in compression should be expressed as those of an equivalent steel cross-section by dividing the contribution of the concrete component by the relevant modular ratios according to (4). For cross-section properties of a composite section with concrete in tension, this concrete should be ignored except where tension stiffening of concrete is to be considered.

(2) The flexural stiffnesses of a composite cross section are defined as $E_a I_1$, $E_a I_2$ and *mod.* $E_a I_{2,ts}$ where:

E_a is the modulus of elasticity for structural steel

I_1 is the second moment of area of the effective equivalent steel section calculated assuming that the concrete in tension is uncracked and using the relevant load dependent modular ratio according to (4),

I_2 is the second moment of area of the effective equivalent steel section (structural steel, reinforcement and prestressing steel) calculated neglecting concrete in tension but including reinforcement within the effective width.

$I_{2,ts}$ is the second moment of area of the effective equivalent steel section as for I_2 , except that the effect of tension stiffening of concrete between cracks is included.

NOTE: Information about the determination of $E_a I_{2,ts}$ is given in L.4 of annex L.

(3) Unless a more precise method is used, the effects of creep in bridge decks and *rep.* members may be taken into account by using modular ratios n_L for the concrete given in (4) except for longitudinal members with both flanges composite (e.g. some box girders).

(4) The modular ratios depending on the type of loading (subscript L) are given by:
add.

$$n_L = n_0 (1 + \psi_L \phi_t) \quad (4.3)$$

where

n_0 = E_a/E_{cm} is the modular ratio for short time loading, E_a is the modulus of elasticity of structural steel and E_{cm} is the secant modulus of elasticity of the concrete for short-term loading according to 3.1.4.1,

ϕ_t is the creep coefficient $\phi(t, t_0)$ according to 3.1.2.5.5 or annex 1 of ENV 1992-1-1:1991 depending on the age (t) of concrete at the moment considered and the age (t_0) at loading. For shrinkage the age at loading should be assumed as one day. For permanent loads on bridge decks cast in several stages one mean value t_0 may be used for the determination of the creep coefficient $\phi(t, t_0)$. This assumption may also be used for prestressing by imposed deformations, if the age of all of the concrete in the relevant spans at the time of prestressing is more than 14 days,

ψ_L is the creep multiplier depending on the creep and ageing coefficient according to 2.5.5.1 of ENV 1992-1-1:1991 and the cross-section properties of the steel and composite section. For bridges and bridge members within the scope given

in (3) above constant values for the creep multiplier according to table 4.0 may be used.

Table 4.0 Values for the creep multipliers ψ_L

permanent loads including prestressing by tendons after the shear connection has become effective	1,10
isostatic and hyperstatic effects due to shrinkage, and time dependent hyperstatic effects according to (5)	0,55
prestressing by imposed deformations (e.g. jacking of supports)	1,50

(5) Where the bending moment distribution at the time t_0 is significantly changed by *add.* creep, for example in continuous beams with both composite and non composite spans, the time dependent hyperstatic effects due to creep should be considered.

(6) When prestressing by tendons is carried out on slabs before the shear connection has *add.* become effective, the sequence of construction and the creep coefficient from the time when the shear connection has become effective should be taken into account.

4.3 Classification of cross-sections of beams

4.3.1 General

(4)P does not apply

(6)P to (10) do not apply.

(11)P For stiffened plates ENV 1993-1-5:1997 is applicable.
add.

(12) In verifications for execution, account should be taken of the class of the steel section
add. at the time considered.

4.3.2 Classification of steel flanges in compression

(2)P The classification of steel flanges in compression in composite beams shall be *mod.* in accordance with table 5.3.1 (sheet 3) of ENV 1993-1-1:1992 and table 4.1, for outstand flanges, and table 5.3.1 (sheet 2) of ENV 1993-1-1:1992, for internal flange elements.

4.3.3 Classification of steel webs

rep.

(1)P The class of the web shall be determined from table 4.2. The plastic stress distribution for the effective composite section shall be used; except at the boundary between Classes 3 and 4, where the elastic stress distribution shall be used.

(2) The distribution of stresses should be determined for the gross cross-section of the steel web and the effective flanges, but neglecting concrete in tension and tension stiffening, and taking into account sequence of construction and the effects of creep and shrinkage.

4.4 Resistance of cross-sections of beams

4.4.1 Bending moment

4.4.1.1 Basis

rep.

(1)P If the section is asymmetrical the effect of asymmetry shall be considered in calculations.

(2)P The design bending resistance may be determined by plastic theory only where the effective composite section is in Class 1 or Class 2.

(3)P Elastic analysis according to 4.4.1.4 and non linear analysis according to 4.4.1.3 may be applied to cross-sections of any class.

(4)P The tensile strength of concrete shall be neglected. It may be assumed that the composite cross-section remains plane, if the shear connection is in accordance with Section 6.

(5)P Clause 4.4 is applicable to composite sections with fully bonded internal tendons and permanently unbonded tendons. Forces in unbonded tendons shall be treated as external actions. It is generally necessary to take into account the deformations of the whole member for the determination of forces in permanently unbonded tendons. Plastic theory in accordance with 4.4.1.2 shall not be used for sections prestressed by tendons.

(6)P Fastener holes in steel elements shall be considered in accordance with ENV 1993-1-1:1992.

(7) Small holes in steel through which reinforcing bars pass should be treated as holes for fasteners.

(8) Local buckling of reinforcing bars that are assumed to contribute to resistance in compression should be prevented by transverse reinforcement.

(9) Isostatic effects of temperature may be neglected in verifications of bending resistance of cross-sections in Class 1 or 2.

4.4.1.2 Plastic resistance moment of a section

rep.

(1)P For composite members with steel elements that are not curved in plan, the plastic resistance moment shall be determined according to (2)P.

(2)P The following assumptions shall be made in the calculation of $M_{pl,Rd}$:

- (a) there is full interaction between structural steel, reinforcement, and concrete;
- (b) the effective area of the structural steel member is stressed to its design yield strength f_y/γ_a in tension or compression;
- (c) the areas of longitudinal reinforcement in tension and in compression within the effective width are stressed to their design yield strength f_{sk}/γ_s in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected.

(3)P It shall be assumed that the effective area of concrete in compression resists a stress of $f_{cd} = 0,85 f_{ck}/\gamma_c$, constant over the whole depth between the plastic neutral axis and the most compressed fibre of the concrete.

(4) Where the distance z_{pl} between the plastic neutral axis and the extreme fibre of the concrete slab in compression exceeds 15% of the overall depth h of the member, the resistance to bending should be determined from 4.4.1.3 or 4.4.1.4. Alternatively the design resistance moment should be taken as $M_{Rd} = \beta M_{pl,Rd}$ where β is the reduction factor given in figure 4.4(*rep.*). For values z_{pl}/h greater than 0,4 the resistance to bending should be determined from 4.4.1.3 or 4.4.1.4.

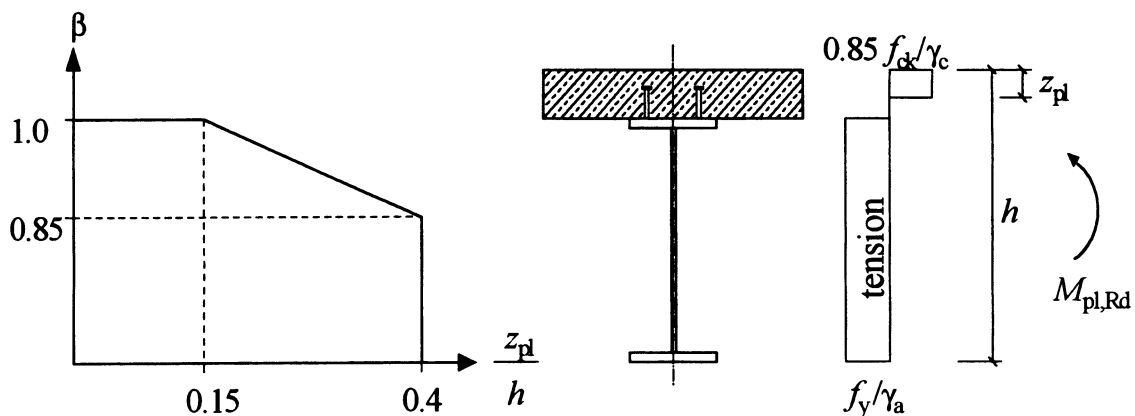


Figure 4.4 (*rep.*): Reduction factor β , cross section and stress distribution

(5) Paragraphs (1) to (4) also apply for the calculation of $M_{f,Rd}$ in 2.2.3.1 and 4.3.4 of ENV 1993-1-5:1997. The resistance of the flanges should be taken as the total strength of the steel, concrete and reinforcement, as applicable. For cross-sections where para (4) applies the same value of β should be used for the calculation of $M_{f,Rd}$ as for $M_{pl,Rd}$.

4.4.1.3 Non-linear resistance to bending

rep.

(1) A non-linear theory for bending resistance taking into account the stress-strain relationships of the materials may be applied for composite sections of any class and with or without prestressing by tendons.

(2) It should be assumed that the composite section remains plane and that the strain in bonded reinforcement, whether in tension or compression, is the same as the mean strain in the surrounding concrete. Effects of tension stiffening of concrete may be neglected.

(3) The stresses in the concrete in compression should be derived from the stress-strain curve in 4.2.1.3.3 of ENV 1992-1-1:1991.

(4) The stresses in the reinforcement or prestressing steel should be derived from the design curves in 4.2.2.3 of ENV 1992-1-1:1991. Reinforcement in compression should be prevented from buckling according to 4.4.1.1(8). The design initial pre-strain in prestressing tendons should be taken into account when assessing the stresses in the tendons.

(5) The stresses in structural steel in compression or tension should be derived from the stress-strain relationship in 3.3.4 and should take account of the effects of the method of construction (e.g. propped or unpropped). For sections in Class 3 and 4 the strain in the extreme compression fibre of the steel section should be limited to $f_y / (E_a \gamma_a)$.

4.4.1.4 Elastic resistance to bending

rep.

(1)P Stresses shall be calculated by elastic theory, using an effective cross section in accordance with 4.2.1, 4.2.2.2 and ENV 1993-1-5:1997. The analysis may be based either on the calculated stresses for the composite section, or the internal forces and moments for the structural steel section and the reinforced or prestressed concrete section separately.

(2) Unless a more precise method is used the effect of creep should be taken into account by use of modular ratios according to 4.2.3 (3) and (4).

(3)P In the calculation of the elastic resistance to bending based on the effective cross-section the limiting stresses shall be taken as:

$0,85 f_{ck}/\gamma_c$ in concrete in compression;

f_y/γ_a in structural steel in tension, or in compression in a cross-section in Class 1, 2, or 3;

- f_y/γ_{Rd} in structural steel in compression in an effective cross-section in Class 4, where $\gamma_{Rd} = \boxed{1,10}$;
- f_{sk}/γ_s in reinforcement in tension or compression. Alternatively, reinforcement in compression in a concrete slab may be neglected;
- $f_{p0,1k}/\gamma_p$ in prestressing tendons according to 2.5.4.4.3 of ENV 1992-1-1:1991. The stress due to initial prestrain in prestressing tendons should be taken into account in accordance with 4.3.1.2 of ENV 1992-1-1:1991.

NOTE: When the effect of tension stiffening of concrete is taken into account, information about the calculation of stresses is given in annex L.

(4)P Stresses due to actions on the structural steelwork alone shall be added to stresses due to actions on the composite member.

(5) In sections with concrete in tension and assumed to be cracked the stresses due to primary (isostatic) effects of shrinkage may be neglected.

4.4.2 Vertical shear

rep.

(1)P The resistance to vertical shear shall be taken as the resistance of the structural steel section in accordance with ENV 1993-1-5:1997 and ENV 1993-2:1997 unless the value of a contribution from the reinforced concrete part of the beam has been established.

(2)P No account shall be taken of a contribution from the concrete slab to the anchorage of a web tension field in a flange, unless the shear connection is designed for the relevant vertical force.

4.4.3 Bending, axial force and vertical shear

rep.

(1)P For a cross-section in Class 1 or 2, allowance shall be made for the effect of a shear force according to 5.4.9 of ENV 1993-1-1:1991.

(2)P For cross-sections in Class 3 and 4, ENV 1993-2:1997 is applicable using either the calculated stresses of the composite section or the sectional forces of the steel cross-section.

(3) When applying 4.3.4 of ENV 1993-1-5:1997 for a beam with one flange composite, the dimension of the non-composite flange may be used even if that is the larger steel flange. The axial normal force N_{sd} in 4.3.4(2) of ENV 1993-1-5:1997 should be taken as the axial force acting on the composite section.

4.4.4 Shear buckling resistance (does not apply)

4.4.5 Interaction between bending and shear buckling (does not apply)

NOTE: The subjects of 4.4.4 and 4.4.5 and other types of plate buckling are covered in ENV 1993-1-5:1997.

4.4.6 Flange-induced buckling of webs

(1) The provisions in 4.4.6 of ENV 1993-1-5:1997 are applicable with the following modification. The area A_{fc} in 4.4.6 of ENV 1993-1-5:1997 should be taken greater than the area of the tension flange, or the transformed area if that flange is composite.

4.5 Global analysis for bridge structures

4.5.1 General *mod.*

(1)P The calculation model and basic assumptions for the calculations shall represent the structural response at the ultimate limit state.

(2)P Unless a more accurate analysis is done, the internal forces and moments shall be determined using either elastic global analysis (4.5.3) or non-linear analysis (4.5.4).

(3)P The influence of deformations shall be considered, where their effects are significant, in accordance with 2.3.2.1(4).

(4)P The effects of slip and uplift may be neglected at interfaces between steel and concrete at which shear connection is provided in accordance with Section 6.

4.5.2 Plastic analysis (does not apply)

4.5.3 Elastic analysis

4.5.3.1 General *rep.*

(1)P Elastic global analysis shall be based on the assumption that the moment-curvature relationships for the sections are linear, whatever the stress level.

4.5.3.2 Sequence of construction

(1)P Appropriate global analysis shall be made to cover the effects of staged *mod.* construction including the separate effects of actions applied to the structural steel and to wholly or partially composite members.

4.5.3.3 Effects of creep and shrinkage of concrete, and of temperature *rep.*

(1)P Account shall be taken of the secondary bending moments (hyperstatic effects) caused by shrinkage and creep of the concrete slab.

(2) Unless a more precise method is used the influence of creep of concrete may be taken into account by use of modular ratios according to 4.2.3(3) and (4).

(3) In regions where the concrete slab is assumed to be cracked the primary (isostatic) effects due to shrinkage may be neglected in the calculation of secondary (hyperstatic) effects.

4.5.3.4 Effects of cracking of concrete *rep.*

(1)P Allowance shall be made for the effects of cracking of concrete.

(2) For continuous beams, including longitudinal beams in multiple-beam decks with the concrete slab above the steel beam, a simplified method in accordance with 4.5.3.4 (3) or (4) may be used where the sensitivity of the results of a global analysis to the extent of cracking of concrete is low. The method used for allowing the effects of cracking should be applied consistently throughout the structure.

(3) Hogging bending moments in the composite member at internal supports calculated by uncracked analysis may be reduced by amounts not exceeding 10%. For each load case, the internal forces and moments after redistribution should be in equilibrium with the loads. For the ultimate limit state of fatigue for Categories A, B and C according to 4.4.0.3 of ENV 1992-2:1996 no redistribution is allowed.

(4) For bridges not prestressed by tendons or imposed deformations (e.g. jacking of supports) where all ratios of the length of adjacent continuous spans (shorter/longer) between supports are at least 0,6, the effect of cracking of concrete may be taken into account by using the flexural stiffness $E_a I_2$ according to 4.2.3 (2) over 15% of the span on each side of each internal support, and as the uncracked values $E_a I_1$ elsewhere. This does not apply to verifications during erection.

(5) Whether 4.5.3.4(2) is applicable or not, the methods (a) or (b) below may be used for the determination of the cracked regions and the bending stiffness in these regions. The internal forces should be calculated using the flexural stiffness of the uncracked sections. For structures with the concrete slab above the steel beam, the bending moment envelope to be used should be that for the infrequent combination of actions including long term effects, and with variable traffic actions assumed to act only on the two spans adjacent to the relevant internal support. For the determination of the bending moment envelope the uniformly distributed traffic loads according to ENV 1991-3:1995 should be arranged simultaneously in both spans adjacent to the relevant support.

(a) In regions where the extreme-fibre tensile stress in the concrete slab due to global effects exceeds the strength $1,3 f_{ctk,0.95}$ according to 3.1.2 the stiffness should be reduced to $E_a I_2$, according to 4.2.3. This distribution of stiffness may be used for global analysis for ultimate limit states, and for serviceability limit states as specified in Section 5.

(b) In regions where the extreme-fibre tensile stress in the concrete slab due to global effects exceeds the tensile strength $f_{ctk,0.95}$ the stiffness should be reduced to $E_a I_{2,ts}$

according to 4.2.3. This distribution of stiffness may be used for global analysis for ultimate limit states, and for serviceability limit states as specified in Section 5.

(6) Unless a more precise method is used, in multiple beam decks the transverse members may be assumed to be uncracked throughout.

4.5.3.5 Prestressing by tendons *add.*

(1) Internal forces due to prestressing by bonded tendons should be determined in accordance with 2.5.4 and 4.2.3 of ENV 1992-1-1:1991. Members containing permanently unbonded tendons should be designed in accordance with ENV 1992-1-6:1994.

4.5.4 Non-linear global analysis *add.*

(1)P A non-linear global analysis shall satisfy the principles of clause 4.1. The following effects shall be considered:

- non-linear behaviour caused by yielding of structural steel, reinforcement and prestressing steel,
- non-linear effects caused by creep, shrinkage and cracking of concrete including tension stiffening of concrete between cracks,
- the load/slip behaviour of the shear connection,
- effects caused by buckling,
- sequence of construction.

NOTE: No application rules are given for these methods.

4.6 Lateral-torsional buckling of composite beams *rep.*

4.6.1 General

(1)P Lateral stability of steel flanges in compression shall be verified unless the flange is attached to the concrete slab by shear connectors in accordance with Section 6.

(2) Where the non-dimensional slenderness $\bar{\lambda}_{LT}$ according to (3) or 4.6.2 does not exceed 0,4 no allowance for lateral torsional buckling is necessary.

(3) The methods of 5.5.2 and 5.5.4.3.1 of ENV 1993-2:1997 are applicable on the basis of the stresses of the composite section, or the cross-sectional forces of the steel section and the properties of the steel section alone, assuming a lateral and elastic torsional restraint at the level of the connection to the concrete deck. For cross-sections in Class 1 or 2 the simplified method given in 4.6.2 may be used.

4.6.2 Lateral buckling of beams with cross sections in Class 1 or 2

(1) The design buckling resistance moment should be taken as

$$M_{b,Rd} = \chi_{LT} M_{Rd} \quad (4.4)$$

where

χ_{LT} is the reduction factor for lateral-torsional buckling according to 5.5.2 of ENV 1993-2:1997, depending on the non-dimensional slenderness $\bar{\lambda}_{LT}$ according to (2),

M_{Rd} is the plastic design resistance moment $M_{pl,Rd}$ given by 4.4.1.2 or the non-linear design resistance to bending according to 4.4.1.3.

(2) The non-dimensional slenderness $\bar{\lambda}_{LT}$ may be determined from

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_{R,k}}{M_{cr}}} \quad (4.5)$$

where

$M_{R,k}$ is the plastic resistance moment or the non-linear resistance moment of the composite section using the characteristic material properties; that is, the value of M_{Rd} according to 4.6.2(1) when the γ_M factors γ_a, γ_c and γ_s are taken as 1,0,

M_{cr} is the elastic critical moment of the composite section for lateral torsional buckling. Where applicable, the method given in B.1.2 to B.1.4 of annex B of ENV 1994-1-1:1992 may be used for the calculation of M_{cr} .

4.6.3 Effects of transverse frames

(1) The buckling resistance of a compressed flange that is laterally restrained by additional transverse frames between internal supports may be determined in accordance with 5.5.2.4 (4) of ENV 1993-2:1997.

(2) The design transverse forces for the stabilising frames should be calculated in accordance with 5.5.2.4 (4) of ENV 1993-2:1997.

4.7 Tension members in composite bridges

rep.

4.7.1 General

(1) In this clause 4.7 the term composite system refers to structures in which shear connection applies global tensile force to a reinforced or prestressed concrete or a composite member. Typical examples are bowstring arches where the concrete or composite member acts as a tension chord in the main system.

(2) Composite tension members are members such as diagonals in tension in trusses or ties in bowstring arches, consisting of structural steel, concrete and reinforcement with a shear connection in accordance with Section 6.

4.7.2 Concrete tension members

(1)P A concrete tension member in a composite system shall be designed in accordance with ENV 1992-1-1:1991. For prestressing by tendons the effect of different bond behaviour of prestressing and reinforcing steel shall be taken into account in accordance with the principles of 4.4.2 of ENV 1992-2:1996.

(2)P For the determination of the forces in the concrete tension member the effects of cracking of concrete and tension stiffening of concrete between cracks shall be considered for the ultimate and serviceability limit state and for checking fatigue.

NOTE: Information about the effects of tension stiffening of concrete between cracks is given in clause L.2 of annex L

(3) The effects of tension stiffening of concrete between cracks may be neglected, if in the global analysis the internal forces of the concrete element are determined with the uncracked stiffness of the concrete tension member, and the internal forces of the structure are determined with the cracked stiffness (neglecting tension stiffening of concrete) of the concrete tension member.

(4) For the calculation of the internal forces of the concrete tension member shrinkage of concrete should be taken into account. Unless a more precise method is used the shrinkage strains valid for the uncracked concrete member according to ENV 1992-1-1:1991 should be used.

(5) For tension members in half-through bridges or bowstring arch bridges where the tension member is simultaneously acting as a deck and subjected to combined global and local effects, the design shear resistance for local vertical shear and for punching shear due to permanent loads and traffic loads should be determined in accordance with 4.3.2.3 of ENV 1992-1-1:1991 by taking into account the normal force N_{sd} of the reinforced concrete element according to 4.7.2 (2) or (3).

(6) At the ends of concrete tension members, for the introduction of a normal force a concentrated group of shear connectors designed according to Section 6 should be provided. The shear connection should be able to transfer the design value of the normal force of the concrete tension element over a length $1,5 b$ where b is the larger of the outstand of the concrete member and half the distance between adjacent steel elements. Where the shear connectors are verified for a normal force determined by (2), an additional partial safety factor $\gamma_F = 1,25$ should be applied, to take into account the uncertainties of the tensile strength of concrete.

4.7.3 Composite tension members

(1)P For ultimate limit states other than fatigue, composite tension members shall be designed for resistance to bending and tension disregarding the tensile strength of concrete. The design should be in accordance with the principles of clause 4.4.

(2)P For the ultimate limit state of fatigue the stresses in reinforcement shall be determined taking into account the effect of tension stiffening of concrete.

NOTE: Information about the effects of tension stiffening of concrete between cracks is given in clause L.2 of annex L.

(3) The effects of tension stiffening of concrete between cracks may be neglected, if sectional forces of the reinforced concrete element are determined with the uncracked stiffness of the composite member, and the sectional forces of the structural section are determined with the cracked stiffness (neglecting tension stiffening of concrete) of the composite member.

(4) For composite tension members subjected to tension and bending a shear connection should be provided in accordance with Section 6.

(5)P Provision shall be made for internal forces and moments applied from members connected to the ends to be distributed between the structural steel and reinforced concrete components, considering the shear connection at the interface between structural steel and concrete.

(6) For composite tension members such as diagonals in trusses, the introduction length for the shear force should not be assumed to exceed twice the minimum transverse dimension of the composite tension member.

4.8 Composite compression members

4.8.1 Scope

(4)P Unless otherwise specified, composite compression members may be used in *add.* bridges.

NOTE: The rules in clause 4.8 of ENV 1994-1-1:1992 have been developed primarily for buildings and the experience from use in bridges is limited.

4.8.2 General method of design

4.8.2.5 Cover and reinforcement

(2) The concrete cover to a flange of a fully-encased steel section should be not less *mod.* than 50 mm nor less than one-sixth of the breadth b of the flange. The cover to reinforcement should be in accordance with 4.1.3.3 of ENV 1992-2:1996.

4.8.2.6 Shear between the steel and concrete components

(1)P For bridges, 4.8.2.7 is applicable only for accidental load situations.
rep.

(3) For composite compression members in bridges the axial force from persistent *rep.* load situations should be introduced to the concrete by direct bearing and/or by mechanical shear connection over a length not exceeding twice the smaller transverse dimension.

4.8.3 Simplified method of design

4.8.3.1 Scope

(3) (c) The non-dimensional slenderness $\bar{\lambda}$ defined in 4.8.3.7 should not exceed 1,5.
mod.

(3) (f) does not apply.

4.8.3.13 Resistance of members in compression and uniaxial bending

(6) Where the acting normal force N_{sd} and bending moment M_{sd} are independent *rep.* the increase in bending resistance due to the normal force may be overestimated. This should be taken into account by the use of load factors according to 2.3.3.1 (103) of ENV 1992-2:1996.

4.9 Internal forces and moments in frames for buildings (does not apply)

4.10 Composite connections in braced frames for buildings (does not apply)

4.11 Box girders

add.

(1)P Clause 5.2.3.3 of ENV 1993-2:1997 is applicable with the additions in (2) to (6). For global analysis, the bending stiffness of the concrete slab in the longitudinal direction shall be according to the principles of 4.5. In the transverse direction the uncracked stiffness may be used.

- (2) The torsional stiffness should be calculated for a transformed cross section in which the slab thickness is reduced by the ratio between the shear moduli of steel and concrete and this area is assumed to be located in the centre of the slab. The shear modulus for concrete may be taken from ENV 1992-1-1:1991 or for simplicity the ratio between the shear moduli may be assumed to be the same as that between the elastic moduli. If the membrane shear stress is so large that shear reinforcement is required, the calculation should be performed considering a slab thickness reduced to one half, unless the effect of cracking is considered in a more precise way.
- (3) The effects of distortion should be taken into account in accordance with 5.2.3.3 of ENV 1993-2:1997.
- (4) The longitudinal shear force on the connectors should include the effects of bending, St. Venant torsion and also distortion if this is not negligible according to (3). For closed top boxes with the top plate designed as a composite plate, see clause 7.7.
- (5) An open top box girder may be provided with profiled sheeting that serves as a part of the structure during erection only, provided that its resistance and fastening are verified according to ENV 1993-1-3:1996.

4.12 Fatigue *add.*

4.12.1 General

- (1)P The resistance of composite road and railway bridges to fatigue shall be verified.
- (2) The verification of reinforced and/or prestressed concrete members, should follow 4.3.7 of ENV 1992-2:1996.
- (3) The verification of steel members should follow Section 9 of ENV 1993-2:1997.
- (4) The verification of shear connectors should follow 6.1.5.

4.12.2 Fatigue loading and partial safety factors

- (1)P The fatigue load models shall follow the clauses 4.6 and 6.9 of ENV 1991-3:1995.
- (2) For road bridges simplified methods according to ENV 1992-2:1996 and ENV 1993-2:1997, based on the Fatigue Load Model 3 of clause 4.6 of ENV 1991-3:1995 may be used for verifications of fatigue resistance.
- (3) The partial safety factors for fatigue loads should be taken from 9.3 (1)P of ENV 1993-2:1997.

(4) The partial safety factors for fatigue resistance should be taken from 9.3 (2) of ENV 1993-2:1997.

(5) For concrete, reinforcement and tendons 4.3.7.2 and appendix 106 of ENV 1992-2:1996 are applicable.

4.12.3 Internal forces

(1)P Internal forces shall be determined for the fatigue load model specified by the relevant authority.

(2) For the simplified assessment according to 4.12.6(1) the maximum and minimum bending moments due to the relevant fatigue load models of ENV 1991-3:1995 are given by

$$M_{\max.f.E} = M_{\text{perm}} + \lambda M_{\max.f} \quad (4.6)$$

$$M_{\min.f.E} = M_{\text{perm}} + \lambda M_{\min.f} \quad (4.7)$$

where:

M_{perm} is the most adverse bending moment in the composite section for the infrequent combination neglecting traffic loads,

$M_{\max.f}$ is the maximum bending moment due to the fatigue load model of ENV 1991-3:1995,

$M_{\min.f}$ is the minimum bending moment due to the fatigue load model of ENV 1991-3:1995,

λ is the correction factor according to (3) and (4) to calculate the damage equivalent stress range from the stress range caused by fatigue load model 3 for road bridges or load model 71 for railway bridges.

(3) For reinforcing and prestressing steel $\lambda = \lambda_s$ should be determined according to clauses A.106.2 and A.106.3 of ENV 1992-2:1996. For road bridges the factored fatigue load model 3 according to clause A.106.2, P(101) of ENV 1992-2:1996 should be used.

(4) For the verification of structural steel λ represents $\Phi_2 \lambda_1 \lambda_2 \lambda_3 \lambda_4$ and should be determined according to 9.4(5), 9.5.1 and 9.5.2 of ENV 1993-2:1997.

4.12.4 Stresses and stress range $\Delta\sigma_E$

- (1) The calculation of stresses should be based on the assumptions given in 5.1.4.1.
- (2)P For concrete flanges in tension and prestressed by tendons the different bond behaviour of prestressing and reinforcing steel shall be taken into account.
- (3) Cracking of concrete should be taken into account in accordance with 5.1.4.2.
- (4) For the calculation of the stress range $\Delta\sigma_E$ according to 4.12.6 (1) in the reinforcing and/or prestressing steel of cracked sections the effect of tension stiffening should be taken into account.

NOTE: Information about the determination of stresses taking into account the effects of tension stiffening is given in clause L.5 of annex L.

- (5) For simplification (alternatively to (4)) the stresses in reinforcement and in tendons may be calculated according to (7) and (8).
- (6) For the calculation of the stress range $\Delta\sigma_E$ according to 4.12.6 (1) in the structural steel of cracked sections the effect of tension stiffening may be taken into account. For simplification for cracked sections according to (3) the stresses in structural steel may be determined by neglecting the effects of tension stiffening of concrete and using the second moment of area I_2 according to 4.2.3.
- (7) In regions where the global bending moments $M_{\max,f,E}$ and $M_{\min,f,E}$ cause tension in the concrete slab the stresses in reinforcement and tendons are given by:

$$\sigma_{\max,f,E} = \sigma_{s,\max,EC} \frac{M_{\max,f,E}}{M_{\max,EC}} \quad (4.8)$$

$$\sigma_{\min,f,E} = \sigma_{s,\max,EC} \frac{M_{\min,f,E}}{M_{\max,EC}} \quad (4.9)$$

where:

$\sigma_{s,\max,EC}$ is the stress in the reinforcement or tendons due to $M_{\max,EC}$ determined according to 5.3.3.1(2)

$M_{\max,EC}$ is the bending moment in the composite section for the infrequent load combination including traffic loads according to the Sections 4 and 6 of ENV 1991-3:1995,

and $M_{\min,f,E}$ and $M_{\max,f,E}$ are defined in 4.12.3(2).

(8) If $M_{\min,f,E}$ according to 4.12.3(2) causes compression in the concrete slab the stresses $\sigma_{\min,f,E}$ in reinforcement, tendons and structural steel should be determined with the cross-section properties of the uncracked section using the modular ratio for short term loading for the bending moment $M_{\min,f}$ as defined in 4.12.3(2).

(9) The stress ranges due to global effects may be calculated from:

$$\Delta\sigma_{E, \text{glob}} = |\sigma_{\max,f,E} - \sigma_{\min,f,E}| \quad (4.10)$$

where:

$\sigma_{\max,f,E}$ is the maximum stress due to the bending moment $M_{\max,f,E}$ according to 4.12.3(2),

$\sigma_{\min,f,E}$ is the minimum stress due to the bending moment $M_{\min,f,E}$ according to (6) or (8) for structural steel, and according to (7) and/or (8) for reinforcement and tendons.

(10) Where a member is subjected to combined global and local effects the separate effects may be combined using:

$$\Delta\sigma_E = \Delta\sigma_{E, \text{glob}} + \lambda_{\text{loc}} \Delta\sigma_{\text{loc}} \quad (4.11)$$

where

$\Delta\sigma_{E, \text{glob}}$ is the damage equivalent stress range due to global effects according to (9)

λ_{loc} is the correction factor according to 4.12.3(3) and (4) for local effects

$\Delta\sigma_{\text{loc}}$ is the stress range caused by local effects

(11) For the calculation of the stresses in the shear connectors see 6.1.5 (1) and 6.2.2.

4.12.5 Fatigue resistance

(1) For concrete in compression, fatigue verifications are not necessary for road bridges and pedestrian bridges, if 5.2(3) is satisfied. For railway bridges see 4.3.7.4 of ENV 1992-2:1996.

(2) The fatigue strengths of reinforcing steel and prestressing steel should be taken from 4.3.7.7 and 4.3.7.8 of ENV 1992-2:1996.

(3) For the fatigue strength of structural steel see clause 9.6 of ENV 1993-2:1997.

(4)P For the fatigue strength of shear connectors see 6.1.5 and 6.3.8.

4.12.6 Simplified assessment

(1) The following expression should be satisfied for the verification of structural steel, reinforcement and prestressing steel.

$$\gamma_{Ff} \Delta \sigma_E \leq \frac{\Delta \sigma_{Rk} (N^*)}{\gamma_{Mf}} \quad (4.12)$$

where:

- γ_{Ff} is given in 4.12.2 (3),
- γ_{Mf} is given in 4.12.2 (4) and (5),
- $\Delta \sigma_E$ is the damage equivalent stress range according to 4.12.4.
- $\Delta \sigma_{Rk} (N^*)$ is the characteristic stress range at N^* cycles from the appropriate S-N lines where for reinforcement and prestressing steel $\Delta \sigma_{Rk} (N^*)$ is equal to $\Delta \sigma_{RSk} (N^*)$ in accordance with clause 4.3.7.5 of ENV 1992-2:1996. For structural steel $\Delta \sigma_{Rk} (N^*)$ is the fatigue strength for $2 \cdot 10^6$ cycles. See clause 9.6 of ENV 1993-2:1997.

(2) The simplified fatigue assessment for connectors is given in 6.1.5.

(3) The verification for concrete in compression should follow 4.3.7.4 of ENV 1992-2:1996.

(4) For reinforcement 4.3.7.5 (101) and (102) of ENV 1992-2:1996 are applicable.

5 Serviceability limit states *rep.*

5.1 General

5.1.1 Scope

- (1) This Section covers the common serviceability limit states, as listed in 2.2.1.1 (6).

5.1.2 Classification of structures

(1)P The composite bridge or specific parts of it shall be classified into environmental classes according to 4.4.0.3 (101)P of ENV 1992-2:1996.

(2) In order to ensure the performance required, the bridge or parts of the bridge may be classified into design categories for serviceability verification, according to 4.4.0.3 of ENV 1992-2:1996. The appropriate category should be agreed with the client. 4.4.0.3 of ENV 1992-2:1996 applies to both the construction phases and to persistent situations.

(3) The combinations of actions for the following verifications of the serviceability limit states should be taken from 4.4.0.3 of ENV 1992-2:1996 unless otherwise specified in the relevant clause.

5.1.3 Global analysis for serviceability limit states

(1) Effective sections should be in accordance with 4.2.1 for serviceability limit states.

(2) Internal forces and moments should be determined by elastic global analysis according to 4.5.3.

(3) The combinations of actions according to 5.1.2 (2) should be used.

(4)P For sequence of construction see 4.5.3.2 (1)P.

(5) The effects of creep and shrinkage of the concrete slab should be considered in accordance with 4.5.3.3.

(6) The effects of cracking of concrete should be taken into account according to 4.5.3.4.

(7) For bridges prestressed by tendons or imposed deformations no redistribution of the bending moment should be made following 4.5.3.4 (3).

(8)P The effects of temperature and differential temperature in accordance with ENV 1991-2-5:1997 shall be considered in verifications.

(9) If during concreting and hardening of the deck the temperature in the steel top flange due to extreme climatic conditions is very low additional differential temperature should be considered.

5.1.4 Calculation of stresses in cross sections

5.1.4.1 General

(1)P Calculation of stresses at the serviceability limit state shall take into account the following effects, where relevant:

- shear lag
- creep and shrinkage of concrete
- cracking and tension stiffening of concrete
- prestressing
- sequence of execution
- temperature effects
- settlements of supports.

(2) Shear lag may be taken into account according to 4.2.2.2.

(3) In the absence of a more rigorous analysis, the effects of creep and shrinkage may be taken into account by using modular ratios, as given in 4.2.3 (4).

(4) Stresses in the concrete slab and its reinforcement caused by simultaneous global and local actions should be added.

NOTE: As long as no ψ -factors are given in ENV 1991-3:1995 for the combination of local and global effects the combination factor $\psi = 1$ may be used.

(5) In cracked sections the isostatic effects of shrinkage may be neglected when verifying stresses.

5.1.4.2 Tension in concrete

(1)P In section analysis tensile strength of concrete shall be neglected.

(2) Unless a more precise method for determining the effect of tension stiffening of concrete between cracks is used, the stresses in reinforcement and prestressing steel may be determined in accordance with 5.3.3.

(3) The influence of tension stiffening on stresses in structural steel may be neglected.

5.2 Limitation of stresses

- (1)P Excessive creep and microcracking shall be avoided by limiting the compressive stress in concrete.
- (2) For concrete prestressed by tendons and/or imposed permanent and/or variable deformations, the maximum compressive stress at transfer of prestressing forces into the deck should be limited to $0,6 f_c(t)$ in accordance with 4.4.1.1 of ENV 1992-2:1996.
- (3) For concrete prestressed by tendons and/or imposed permanent and variable deformations, the compressive stress in the concrete under the infrequent combination of actions and the characteristic value of prestress should be limited to $0,6 f_{ck}$ in accordance with 4.4.1.1 of ENV 1992-2:1996 unless the compression zone is confined by, for example, transverse reinforcement in excess of 1% of the volume of the compression zone.
- (4)P The stress in reinforcing steel and in prestressing tendons shall be such that inelastic strains in the steel are avoided.
- (5) The tensile stress in the ordinary reinforcement under the infrequent combination of actions should not exceed $0,8 f_{sk}$ in accordance with 4.4.1.1 of ENV 1992-2:1996.
- (6) The stress in prestressing tendons at infinite time under the quasi-permanent combination should not exceed $0,65 f_{pk}$ after allowance for all losses in accordance with 4.4.1.1 of ENV 1992-2:1996.
- (7) For precast elements that are the subject of an adequate control system, see Section 7 of ENV 1992-1-1:1991, the value $0,6 f_c(t)$ in accordance with 4.4.1.1 of ENV 1992-2:1996 may be exceeded during construction by 10% if strict control of strength is provided and a check of loss of prestress at the time considered is made.
- (8) The stresses in structural steel under the characteristic combination of actions should be in accordance with clause 4.3 of ENV 1993-2:1997.
- (9) Forces applied to a shear connector should be limited according to 6.1.3 (2).

5.3 Crack and decompression control

5.3.1 General

- (1)P Cracking shall be limited to a level that will not be expected to impair the proper functioning and durability of the structure.
- (2) For design categories A, B and C according to 4.4.0.3 of ENV 1992-2:1996 no tensile stresses in the concrete are allowed under the relevant load combination.
- (3)P Appropriate limits to design crack widths, taking into account the proposed function and nature of the structure, shall be determined.
- (4) Design crack-width limits should be agreed with the client.

(5)P Limitation of cracks to acceptable width, and the avoidance of uncontrolled cracking between widely-spaced bars, shall be achieved by ensuring that, at all sections likely to be subjected to tension due to imposed permanent and variable deformations and/or direct loading, a minimum amount of bonded reinforcement is present, sufficient to ensure that the reinforcement will remain elastic and the crack width will be limited to the design crack width when single cracking occurs.

(6) In the absence of specific requirements, it may be assumed that for exposure classes 2 to 4, according to 4.1.3.3 of ENV 1992-1-1:1991, limitation of the design crack width to 0,3 mm for reinforced concrete and 0,2 mm for composite bridges prestressed by tendons within the concrete slab will generally be satisfactory in respect of durability. Where there is normal reinforcement in the longitudinal direction and prestressing by tendons in the transverse direction the crack widths should be limited to 0,2 mm in the longitudinal direction.

(7) Application rules are given in 5.3.2 and 5.3.3 for design crack widths w_k of 0,3 mm for reinforced concrete and 0,2 mm for prestressed concrete, for general use. For general use the reinforcing bars should have high-bond action, in accordance with 3.2.2 (1). For other design crack widths, reference should be made to 4.4.2.3 of ENV 1992-2 : 1996.

(8) The minimum reinforcement according to 5.3.2 is adequate to control the crack width according to 5.3.1 (7) provided that it can be verified that under the relevant combination of actions for controlling the crack width, the concrete stress in the extreme fibre of the section does not exceed the mean value of the concrete tensile strength ($\sigma_c \leq f_{ctm}$).

(9) Where 5.3.1 (8) is not satisfied 5.3.2 and 5.3.3 should be applied.

(10) Reinforcing bars with diameters smaller than 10 mm should normally not be used.

5.3.2 Minimum reinforcement

5.3.2.1 General

(1) A minimum reinforcement for reasons of durability and appearance should be placed according to 4.4.2.2.1 of ENV 1992-2:1996.

5.3.2.2 Minimum reinforcement for bridges without prestressing by tendons

(1) In determining the minimum area of reinforcement required to ensure that the reinforcement remains elastic when cracking first occurs, account should be taken of the different types of restraint distinguished in 4.4.2.2 of ENV 1992-1-1:1991, and of the stress distribution in the concrete just before it cracks. The minimum reinforcement according to (2) should be placed where under the infrequent combination of actions the reinforced concrete resists tensile stress, and for prestressed concrete the stresses are less than 1 N/mm² compressive.

(2) Where crack widths are to be controlled in a concrete flange of a composite bridge, (and unless more rigorous calculation shows a lesser area to be adequate) at the face subjected to the greater tensile strain, the minimum reinforcement of the effective area of the concrete flange within the tensile zone, A_{ct} , should satisfy the requirements according to 4.4.2.2.3 of ENV 1992-2:1996 with :

$$\rho_s \geq \frac{0,9k_c k f_{ctm}}{\sigma_s} \quad (5.1)$$

where:

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

A_{ct} is the area of the tensile zone immediately prior to cracking of the cross section; for simplicity the area of the concrete section within the effective width should be used,

$$k_c = \frac{1}{1 + \frac{h_c}{2z_0}} + 0,3 \leq 1,0 \quad (5.2)$$

where:

h_c is the depth of the concrete slab,

z_0 is the vertical distance between the centroids of the uncracked unreinforced concrete flange and the uncracked unreinforced composite section, calculated using the modular ratio for short term effects, E_a / E_{cm} .

k should be taken as $k = 0,8$,

f_{ctm} is the mean value of the concrete tensile strength,

σ_s is the steel stress in the minimum reinforcement area according to table 5.1. The stress σ_s is a function of the maximum bar diameter ϕ_s^* given in table 5.1, (see also 5.3.1 (10)).

σ_s may be increased by a factor

$$\eta = \sqrt{f_{ctm} / f^*_{ctm}}$$

where $f^*_{ctm} = 2,5 \text{ N/mm}^2$ and f_{ctm} is the actual tensile strength of the concrete. σ_s should not exceed $k f_{sk}$

(3) At least half of the required minimum reinforcement should be placed between mid-depth of the slab and the face subjected to the greater tensile strain.

Table 5.1 Steel stress - maximum bar diameter ϕ_s^* .

Steel stress σ_s [N/mm ²]	Maximum bar size ϕ_s^* [mm]	
	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	---

5.3.2.3 Minimum reinforcement for bridges with prestressing by bonded tendons

- (1) The contribution of prestressing steel to the limitation of crack width may be taken into account.
- (2) For bridges with prestressing by tendons the minimum reinforcement ratio ρ_s should be:

$$\rho_s = \frac{0,9k_c k_f f_{ctm}}{\sigma_s} - \xi_1 \rho_p \quad (5.3)$$

where:

ξ_1 is the adjusted ratio of bond strength according to 5.3.3.2 (2),

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

5.3.3 Control of cracking

5.3.3.1 Bridges without prestressing by tendons

(1) Tensile stresses in reinforcement should be determined by elastic analysis of cross sections. The effect of tension stiffening in a composite section increases the tensile stress that is relevant to control of cracking to a value σ_s . The tensile stress σ_s may be calculated according to (2).

(2) The tensile stress in reinforcement for bridges without prestressing by tendons may be calculated by :

$$\sigma_s = \sigma_{se} + \frac{0,4 f_{ctm}}{\alpha_{st} \rho_s} \quad (5.4)$$

where:

σ_{se} is the stress in the reinforcement, calculated neglecting concrete in tension,

A_{ct} is the area of the tensile zone immediately prior to cracking of the cross section; for simplicity the area of the concrete section within the effective widths should be used,

A_s is the total area of all layers of longitudinal reinforcement within the effective area A_{ct} ,

$\rho_s = (A_s / A_{ct})$,

f_{ctm} is the mean tensile strength of the concrete,

$\alpha_{st} = AI / (A_a I_a)$,

where A and I are area and second moments of area, respectively, of the composite section neglecting concrete in tension; and A_a and I_a are the corresponding properties of the structural steel section.

(3) The crack widths may be considered to be adequately controlled if either the bar diameter does not exceed the value given in table 5.1, (see also 5.3.1 (10)), or where the steel stress σ_s is within the range available, the maximum bar spacing does not exceed the limits in table 5.2.

Table 5.2 Maximum bar spacing for high bond bars.

Steel stress σ_s [N/mm ²]	160	200	240	280
Maximum bar spacing [mm]	200	150	125	75

5.3.3.2 Bridges with prestressing by bonded tendons

(1) The clauses 5.3.3.1 (1) and (3) apply. The stresses in the reinforcement and in the prestressing steel should be calculated according to (2) taking into account the different bond behaviour of prestressing and reinforcing steel.

(2) The stresses σ_s in reinforcement and the stresses σ_p in the tendons caused by external moments may be calculated by:

$$\sigma_s = \sigma_s^II + 0,4 f_{ctm} \left(\frac{1}{\text{eff } \rho_p} - \frac{1}{\text{eff } \rho_{tot}} \right) \quad (5.5)$$

$$\sigma_p = \sigma_s^II - 0,4 f_{ctm} \left(\frac{1}{\text{eff } \rho_{tot}} - \frac{\xi_1^2}{\text{eff } \rho_p} \right) \quad (5.6)$$

with:

$$\sigma_s^II = \sigma_{se} + \frac{0,4 f_{ctm}}{\text{eff } \rho_p \alpha_{st}} \quad (5.7)$$

where the symbols are defined in 5.3.3.1 (2) and as follows :

σ_s is the stress in the reinforcement caused by external moments,

σ_p is the stress in the tendons caused by the external moments; to this stress the resulting stress from the prestressing force should be added,

with:

$$\text{eff } \rho_p = \frac{A_s + \xi_1^2 A_p}{A_{ct}} \quad \text{eff } \rho_{tot} = \frac{A_s + A_p}{A_{ct}},$$

where:

A_p is the area of the tendons within the effective area A_{ct} ,

ξ_1 is the adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel and may be calculated by : $\xi_1 = \sqrt{\xi \frac{\phi_s}{\phi_p}}$.

If only prestressing steel is used then ξ_1 should be taken as 1,0.

ϕ_s is the largest diameter of steel reinforcement,

ϕ_p is the equivalent diameter of prestressing steel;

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

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

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

ξ is the ratio of mean bond strength of prestressing steel and high bond reinforcing steel. In the absence of appropriate data ξ may be taken from table 5.3 :

Table 5.3 Nominal ratios ξ for crack control.

Type of tendon	Pretensioned members	Posttensioned 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

5.4 Deformations

(1) For the calculation of deformations either the effective widths according to 4.2.2.1 may be used or alternatively rigorous analysis according to 4.2.1 (1)P.

(2) The tension stiffening effect may be included in calculation of the deformations.

(3)P Elastic analysis according to the principles of 4.5.3 shall be used, and shall take into account the relevant effects according to 5.1.4.1 (1)P.

(4)P Deformations shall not adversely affect the drainage of water from the structure or the use or efficiency of the structure.

(5) For railway bridges, clause G.3 of ENV 1991-3:1995 is applicable. For other bridges, limiting values of deformations in presence of traffic should, where relevant, be agreed with the client, together with the associated combinations of actions.

(6)P Deformations during construction shall be controlled such that the concrete is not impaired during its placing and setting by uncontrolled displacements and the required long-term geometry is achieved.

5.5 Vibration

(1) For the limit state of vibration see clauses 5.7 and 6.4 of ENV 1991-3:1995, 4.4.4 of ENV 1992-2:1996 and clauses 4.7, 4.8 and 4.9 of ENV 1993-2:1997.

6 Shear connection

6.1 General

6.1.1 Basis of design

rep.

(1)P Clauses 6.1 to 6.6 are applicable to beams and box girders and, as appropriate, to other types of member.

NOTE: Supplementary clauses on shear connection are as follows:

clause 4.7 for tension members and bowstring arches

4.8.2.6(3) for compression members

4.11 (4) for box girders

7.7.4 for composite plates.

(2)P Shear connectors and transverse reinforcement shall be provided throughout the length of a member that is designed as composite, to transmit shear force between the concrete element and the structural steel element, except as provided in clause 4.8 for compression members and annex K for filler-beam decks. The use of composite action for both flanges of a composite member is not excluded.

(3)P Shear connectors shall be capable of preventing separation of the concrete element from the steel element, except where separation is prevented by other means, such as the encasement of a steel flange in concrete.

(4) To prevent uplift of the concrete slab, shear connectors should be designed to resist a nominal ultimate tensile force, perpendicular to the plane of the steel flange, of at least 0.1 times the design ultimate shear resistance of the connectors. If necessary, they should be supplemented by anchoring devices.

(5) Headed stud shear connectors in accordance with 6.3.2, 6.4.2, and 6.4.3 may be assumed to have the resistance required by (4) above.

(6) Adjacent to cross frames and vertical web stiffeners, and for composite box girders, the effects should be considered of restraint by the shear connection of rotation of the slab about an axis parallel to the axis of the steel beam. The design should be such that no significant uplift forces are applied to shear connectors.

(7)P Bond (adhesion) between steel and concrete shall not be relied on as shear connection, except as provided in clause 4.8 and annex K.

(8)P Where a method of interconnection, other than the shear connectors included in clause 6.3, is used to transfer shear between a steel element and a concrete element, the behaviour assumed in design shall be based on tests and supported by a conceptual model. The design of the composite member shall conform to the design of a similar member employing shear connectors in accordance with clause 6.3, in so far as practicable.

(9)P Friction (defined as resistance to slip at an interface between steel and concrete, that arises from the prevention of separation, or from compressive stress across that interface) shall

not be assumed to provide shear connection, except where the use of friction is specifically referred to in this Part of ENV 1994.

(10)P Where two or more different types of shear connector are used within the same span of a beam, account shall be taken of any significant difference between their load-slip properties.

(11) In the application of 6.1.1(10)P, stud connectors (6.3.2 and 6.3.3), block connectors (6.3.4 to 6.3.6) and angle connectors (6.3.7) should be assumed to be different types of connector.

6.1.2 Deformation capacity of shear connectors

(1)P Shear connectors shall have sufficient deformation capacity to provide any *mod.* inelastic redistribution of shear between connectors that is assumed in design.

(2) It may be assumed that (1)P is satisfied, in respect of design methods given in this *mod.* Part of ENV 1994, by shear connectors for which resistances are defined in clause 6.3.

(3) and (4) do not apply.

6.1.3 Serviceability limit states

rep.

(1)P The stiffness of the shear connection shall be sufficient to ensure that the influence of longitudinal slip at the interface between steel and concrete on deformations and longitudinal stresses in the member is negligible.

(2) For verifications for the serviceability limit state, the size and spacing of shear connectors may be kept constant over any length where the design longitudinal shear per unit length does not exceed the design shear resistance per unit length by more than 10%, where the design longitudinal shear resistance is given by $0,6 P_{Rk}$, where P_{Rk} is the characteristic resistance of the connector in accordance with 6.3.1(3). Over every such length, the total design longitudinal shear force should not exceed $0,6 P_{Rk}N$, where N is the number of connectors within the length considered.

6.1.4 Ultimate limit states other than fatigue

add.

(1) For verifications for ultimate limit states, 6.1.3(2) is applicable with $0,6 P_{Rk}$ replaced by the design resistance P_{Rd} , defined in 6.3.

(2)P Splitting of the concrete slab due to concentrated forces applied by the connectors shall be prevented.

(3) If the detailing of the shear connection is in accordance with clause 6.4 and the transverse reinforcement is in accordance with clause 6.6, compliance with 6.1.4(2)P may be assumed.

6.1.5 Fatigue assessment based on nominal stress ranges

add.

(1)P For verification of shear connectors based on nominal stress ranges, fatigue loadings and partial safety factors γ_{Ff} shall be in accordance with 4.12.2.

(2) The equivalent constant-amplitude shear stress range for 2 million cycles in welds for studs and other types of shear connector, $\Delta\tau_E$, should be calculated according to Section 9 of ENV 1993-2:1997, except that for stud shear connectors:

- the λ_1 -factors, denoted $\lambda_{v,1}$, should be determined using $m = 8$ instead of $m = 5$;
- in the formulae in 9.5.1 of ENV 1993-2:1997, λ_2 to λ_4 should be calculated using exponents 8 and 1/8 in place of 5 and 1/5, respectively.

(3) For stud connectors in road bridges of span up to 100 m, $\lambda_{v,1} = 1,55$. For stud connectors in railway bridges the $\lambda_{v,1}$ -factor may be taken from figure 6.1 (*rep.*).

(4) Where stud shear connectors are welded to a steel flange that is always in compression under the characteristic combination of actions, the criterion for fatigue assessment is

$$\gamma_{Ff} \gamma_{Mf,v} \Delta\tau_E / \Delta\tau_c \leq 1$$

where $\Delta\tau_E$ is defined in (2) above,

$\Delta\tau_c = 95 \text{ N/mm}^2$, the reference value for $\Delta\tau_R$ for $N_c = 2 \times 10^6$ cycles,

γ_{Ff} is a partial safety factor defined in clause 9.3(1)P of ENV 1993-2:1997, and

$\gamma_{Mf,v}$ is 1,0.

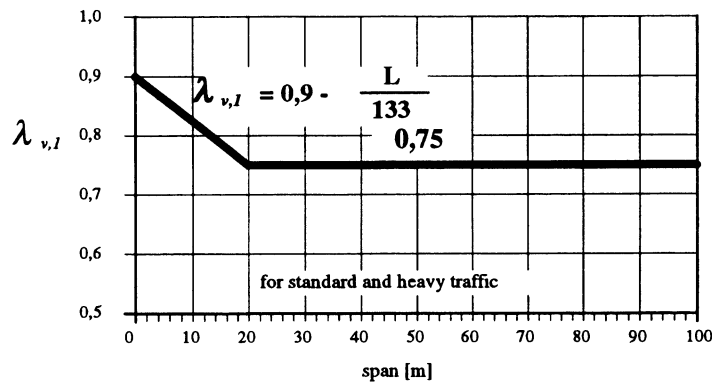


Figure 6.1 (*rep.*): values of $\lambda_{v,1}$ for load model 71, to ENV 1991-3:1995

(5) Fatigue verifications for other types of weld for shear connectors should be in accordance with clauses 9.5 and 9.6 of ENV 1993-2:1997.

(6) Where the maximum stress in a steel flange to which stud connectors are welded is tensile under the infrequent combination of actions including traffic loads according to Sections 4 and 6 of ENV 1991-3:1995, equivalent constant-amplitude stress ranges $\Delta\sigma_E$ in the flange should be calculated according to 4.12.3 and 4.12.4.

(7) Where (6) applies, it should be verified that

$$\gamma_{Ff} \gamma_{Mf,a} \Delta\sigma_E / \Delta\sigma_c \leq 1, \quad \text{and} \quad \gamma_{Ff} \gamma_{Mf,v} \Delta\tau_E / \Delta\tau_c \leq 1, \quad (6.1) \text{ rep.}$$

where $\Delta\tau_c$, γ_{Ff} and $\gamma_{Mf,v}$ are defined in (4) above,

$\gamma_{Mf,a}$ is γ_{Mf} as defined in 9.3(2) of ENV 1993-2:1997, and

$\Delta\sigma_c$ is the reference value of $\Delta\sigma_R$ for $N_c = 2 \times 10^6$ cycles as given in 9.6.2.1 of ENV 1993-1-1:1992 for the relevant detail category.

The interaction at any cross-section between shear stress range $\Delta\tau_E$ in the welds to stud connectors and the normal stress range $\Delta\sigma_E$ in the steel flange should be verified using the following interaction expressions:

$$\gamma_{Ff} [\gamma_{Mf,a} \Delta\sigma_E / \Delta\sigma_c + \gamma_{Mf,v} \Delta\tau_{E,c} / \Delta\tau_c] \leq 1,3 \quad (6.2) \text{ rep.}$$

and
$$\gamma_{Ff} [\gamma_{Mf,a} \Delta\sigma_{E,c} / \Delta\sigma_c + \gamma_{Mf,v} \Delta\tau_E / \Delta\tau_c] \leq 1,3. \quad (6.3) \text{ rep.}$$

where the stress ranges $\Delta\tau_{E,c}$ and $\Delta\sigma_{E,c}$ are the stress ranges corresponding to $\Delta\sigma_E$ and $\Delta\tau_E$ respectively.

(8) To avoid non-linear behaviour caused by fatigue loading, the maximum longitudinal shear per connector should be limited. Where the shear connection is in accordance with 6.1.3, no additional verifications are necessary.

6.1.6 Transient design situations during execution *add.*

(1) When, during execution, actions act on a composite member before the concrete has reached its design strength, the shear connectors should be assumed to be effective provided that the cylinder strength of the concrete surrounding them is not less than 20 N/mm². For strengths less than 20 N/mm², see 9.2(3).

6.2 Longitudinal shear force *rep.*

6.2.1 General

(1) For any combination and arrangement of design actions, longitudinal forces in steel or concrete elements should be determined in accordance with the global analysis for bending moments and vertical shear forces.

(2) The longitudinal shear per unit length of an interface between steel and concrete in a composite member, v_{sd} , should be calculated by elastic theory from the rate of change of the longitudinal force in either the steel element or the concrete element of the composite section, except as provided in 6.2.3. The envelope of transverse shear force in the relevant direction may be used.

(3) In members where cracking of concrete occurs, and account is taken in global analysis of the effects of tension stiffening, then in cracked regions, longitudinal shear per unit length may be determined using either I_1 or $I_{2,ts}$, defined in 4.2.3(2). Elsewhere, the uncracked second moment of area I_1 should be used.

(4) Where 6.2.1(3) does not apply, longitudinal shear per unit length should be determined using the second moment of area I_1 .

(5) Where concentrated longitudinal shear forces occur, account should be taken of the local effects of longitudinal slip; for example, as provided in 6.2.4. Otherwise, the effects of longitudinal slip may be neglected.

(6) Where a sudden change of cross-section leads to an excessive local value for v_{Sd} , and in the absence of a more precise analysis, the distribution along the interface of the longitudinal shear force V_ℓ caused by the change of cross-section may be assumed to be as given in 6.2.4.3, with e_d taken as zero.

6.2.2 Serviceability limit states, and fatigue

(1) Longitudinal forces in steel or concrete elements should be determined for the characteristic combination of actions, in accordance with 5.1.3 and 5.1.4.

6.2.3 Ultimate limit states, other than fatigue, for members in Class 1 or 2

(1) In members with some cross-sections in Class 1 or 2 to clause 4.3, there may be lengths where the design bending moment M_{Sd} exceeds the bending resistance $M_{el,Rd}$, defined by

$$M_{el,Rd} = M_{a,Sd} + k M_{c,Sd},$$

where $M_{a,Sd}$ and $M_{c,Sd}$ are the design bending moments applied to the steel element and the composite element, respectively, at each cross-section; and factor $k (\leq 1)$ has the lowest value such that a stress limit given in 4.4.1.4 is reached at that cross-section. Within these inelastic lengths, shown as ABD in figure 6.2(a), account should be taken of the non-linear relationship between transverse shear and longitudinal shear.

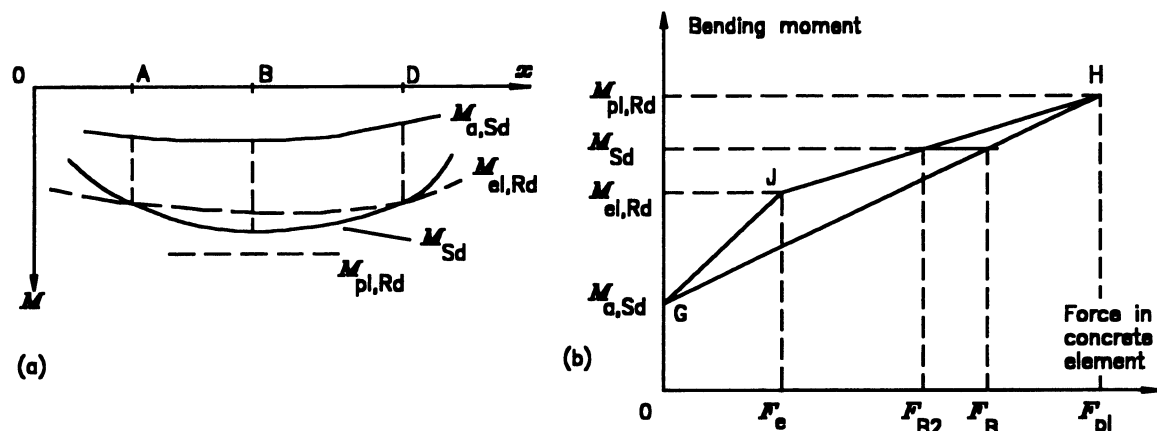


Figure 6.2(rep): longitudinal shear in a beam in Class 1 or 2

(2) The shear connection in length ABD may be verified as follows, for any distribution of bending moments M_{Sd} , where

$$M_{Sd} = M_{a,Sd} + M_{c,Sd}.$$

(a) Let B be the cross-section within ABD where the ratio of M_{Sd} to $M_{pl,Rd}$ (as defined in 4.4.1.2) is a maximum. Let F denote longitudinal forces in the effective reinforced concrete flange, determined by elastic analysis of the composite section, with values:

- F_e at section B when the bending moment is $M_{el,Rd}$, defined above;
- $F_{e,A}$ at section A when the bending moment is $M_{Sd,A}$;
- $F_{e,D}$ at section D when the bending moment is $M_{Sd,D}$.

(b) The numbers of shear connectors provided should be sufficient to resist forces $F_{B2} - F_{e,A}$ within length AB and $F_{B2} - F_{e,D}$ within length BD, where F_{B2} is the force given by line JH in figure 6.2(b), as follows:

$$F_{B2} = F_e + (F_{pl} - F_e) (M_{Sd} - M_{el,Rd}) / (M_{pl,Rd} - M_{el,Rd}),$$

where F_{pl} , M_{Sd} , $M_{el,Rd}$, and $M_{pl,Rd}$, are evaluated at cross-section B. In place of F_{B2} , the more conservative value F_B , given by line GH, may alternatively be used

6.2.4 Local effects of concentrated longitudinal shear force

6.2.4.1 Scope

(1) Clause 6.2.4 is applicable to the determination of the distribution along an interface between steel and concrete of the design longitudinal shear force V_ℓ caused by:

- the isostatic effects of the application of a force F_d to the concrete or steel element by a bonded or unbonded tendon;
- the application of a force F_d to the concrete element of a flange by steel or composite web members of a truss or frame;

where F_d is the component of the relevant force in a direction parallel to the plane of the interface, applied after the shear connection has become effective.

(2) The distribution of V_ℓ caused by several forces F may be obtained by summation.

6.2.4.2 Design longitudinal force for the shear connection

(1) Where a longitudinal force F_d is applied to the concrete element, the design longitudinal force for the shear connection should be evaluated at the final time ($t \rightarrow \infty$), and taken as

$$V_\ell = F_d [(A_a/A) - e(A_{rc} z_{rc} / I_1)];$$

and where the longitudinal force is applied to the structural steel element, the design longitudinal force for the shear connection should be evaluated at the time of loading and taken as

$$V_\ell = F_d [(A_{rc}/A) - e(A_a z_a / I_1)];$$

where:

- A and I_1 are the transformed area and second moment of area, respectively, of the uncracked composite section in 'steel' units;
- A_a and A_{rc} are the cross-sectional areas of the structural steel and reinforced concrete components of the section, respectively, in 'steel' units;
- e is the distance, measured in the plane of bending, from the centroid of the area A to the line of action of the force F_d , positive when downwards;
- z_a and z_{rc} are the distances from the centroid of the area A to the centroids of the areas A_a and A_{rc} , respectively, positive when downwards.

The relevant modular ratio should be used in the calculations.

(2) In the absence of a more precise determination, the force $F_d - V_\ell$ may be assumed to disperse into the concrete or steel element at an angle of spread 2β , where $\beta = \arctan 2/3$.

6.2.4.3 Distribution of the longitudinal force along the interface

(1) The force V_ℓ may be assumed to be distributed along a length L_v of shear connection as shown (for example) in figure 6.3(rep.), with a maximum shear force per unit length given by

$$v_{d,max} = V_\ell / (e_d + b_{eff}/2),$$

where

b_{eff} is the effective width for global analysis, given by 4.2.2.1,

e_d is either $2e_h$ or $2e_v$,

e_h is the lateral distance from the point of application of force F_d to the relevant steel web, if it is applied to the slab,

e_v is the vertical distance from the point of application of force F_d to the plane of the shear connection concerned, if it is applied to the steel element.

Values of the maximum axial forces shown in figure 6.3(b) are given by $\Delta N = v_{d,max} b_{eff}/4$.

(2) Where the force F_d is applied over a length that is not negligible (e.g., through a gusset plate in a composite truss), that length may be added to e_d .

(3) Where stud shear connectors are used, a rectangular distribution of shear force per unit length may be assumed, so that

$$v_{d,max} = V_\ell / (e_d + b_{eff}).$$

(4) Where shear connection is not present along the whole of the length L_v , (e.g., at a free end of a slab), the distribution of v_d should be modified as appropriate.

(5) If the force F_d is applied at a free end of the concrete or steel element, the distribution of the force V_ℓ may be assumed to be half that shown in figure 6.3(a); i.e., extending along a length of shear connection $L_v/2$, with a maximum shear force per unit length given by

$$v_{d,max} = 2 V_\ell / (e_d + b_{eff}).$$

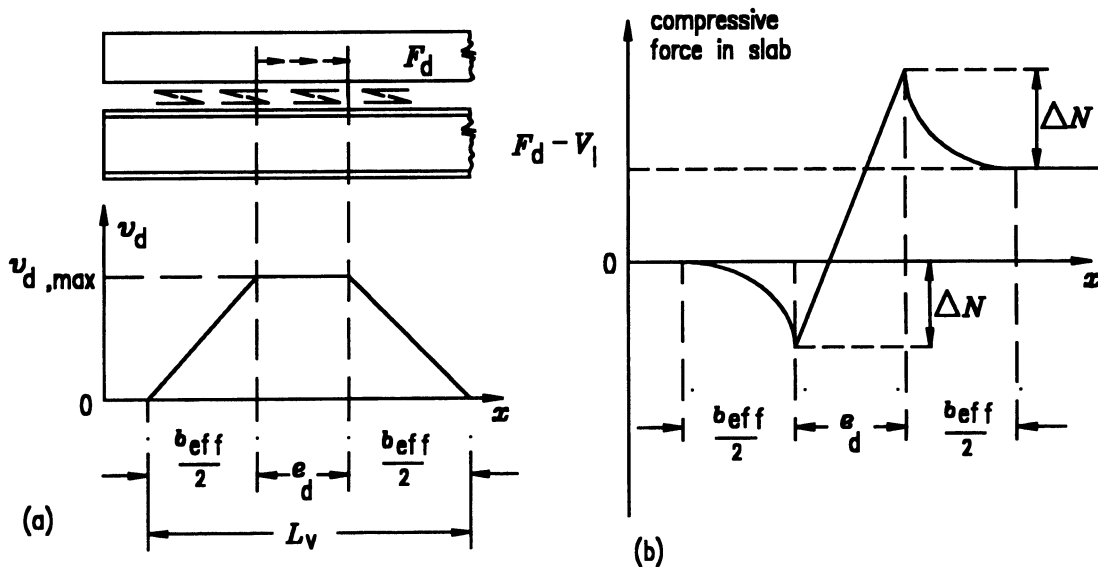


Figure 6.3 (rep.): Distribution of longitudinal shear force along the interface

6.2.5 Temperature effects

(1)P Longitudinal shear due to thermal actions in accordance with ENV 1991-2-5:1997 shall be considered, and taken into account where appropriate.

NOTE: the relevant effects are:

- isostatic (primary) effects due to non-linear variation of temperature through the depth of the composite member;
- isostatic effects caused by a uniform change or linear variation of temperature, where the coefficients of thermal expansion of the steel and concrete are significantly different;
- hyperstatic (secondary) effects in continuous members due to restraint of the deformations caused by the isostatic effects.

(2) Calculations for isostatic effects give a design longitudinal shear force, V_ℓ , to be transferred across the interface between steel and concrete at each free end of the member considered. The distribution of this force may be assumed to be triangular, with a maximum shear force per unit length

$$v_{d,\max} = 2 V_\ell / b_{\text{eff}}$$

at the free end of the slab, where b_{eff} is the effective width for global analysis, given by 4.2.2.1. Where stud shear connectors are used, the distribution may alternatively be assumed to be uniform along a length b_{eff} adjacent to the free end of the slab.

(3) The forces transferred by shear connectors may be assumed to disperse into the concrete slab at an angle of spread 2β , where $\beta = \arctan 2/3$.

6.2.6 Shrinkage modified by creep

(1)P Where the effects of shrinkage of concrete adversely affect the maximum resultant forces on the shear connectors or the maximum resultant stresses in the concrete element, they shall be taken into account where appropriate.

(2) Longitudinal shear caused by shrinkage of concrete may be neglected in continuous spans where all cross-sections of maximum bending moment (including those at internal supports) are in Class 1 or 2. It should not be neglected at free ends of cantilevers.

(3) Where it is necessary to estimate the effects of shrinkage, they may be determined by a method similar to that given for temperature effects in 6.2.5, using a modular ratio that takes account of the reduction of shrinkage effects by creep of concrete in accordance with 4.2.3.

(4) Where the isostatic effects of shrinkage are determined at intermediate stages of the construction of a concrete slab, the breadth b_{eff} in 6.2.4.3 should be determined for an equivalent span equal to the continuous length of concrete slab where the shear connection is effective, within the span considered.

6.3 Design resistance of shear connectors

6.3.1 General

rep.

NOTE: the design resistances given in 6.3.1 to 6.3.7 are for ultimate limit states other than fatigue. For fatigue resistances, see 6.1.5(5) and 6.3.8. For serviceability limit states, see 6.1.3(2).

NOTE: all references to the height of a stud connector refer to the height after welding.

- (1) Where the concrete slab is unhaunched, or the haunch satisfies 6.4.1.4, the design resistance of shear connectors embedded in normal-density concrete, or in lightweight-aggregate concrete of density class not lower than 1,6, should be calculated from the equations given in this clause 6.3.
- (2) Where the concrete density or haunch dimensions do not satisfy the conditions in (1) above, or where the shear connectors are of a type not covered in this clause 6.3; then in the absence of a European Technical Specification the characteristic resistance and the design resistance should be determined from push tests in accordance with clause 10.2.
- (3) The characteristic resistance P_{Rk} of a shear connector should be calculated from the relevant expression for the design resistance P_{Rd} , but with the partial safety factor taken as 1,0.

6.3.2 Stud connectors in solid slabs

6.3.2.1 Headed studs - shear resistance

- (1) The design shear resistance of a headed stud automatically welded in accordance *mod.* with prEN ISO 14555, 'Welding - Arc stud welding of metallic materials' should be determined from

$$P_{Rd} = 0,8 f_u (\pi d^2 / 4) / \gamma_v \quad (6.13)$$

or

$$P_{Rd} = 0,29 \alpha d^2 \sqrt{(f_{ck} E_{cm})} / \gamma_v \quad (6.14)$$

whichever is smaller, where

- d is the diameter of the shank of the stud, as shown in figure 6.3.1;
- f_u is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm²;
- f_{ck} is the characteristic cylinder strength of the concrete at the age considered;
- E_{cm} is the nominal value of the secant modulus of the concrete in accordance with 3.1.4.1;
- $\alpha = 0,2 [(h/d) + 1]$ for $3 \leq h/d \leq 4$;
- $\alpha = 1$ for $h/d > 4$;
- h is the overall height of the stud; and
- γ_v is a partial safety factor to be taken as 1,25 for ultimate limit states other than fatigue.

(2) The formulae in (1) should be verified by tests before being used for:

- add.*
- studs of diameter greater than 25 mm;
 - studs with weld collars smaller than specified in prEN ISO 13918.

NOTE: if no specification for weld collars is given in EN ISO 13918, reference should be made to the ENV NOTE below 6.3.2.1 of ENV 1994-1-1:1992.

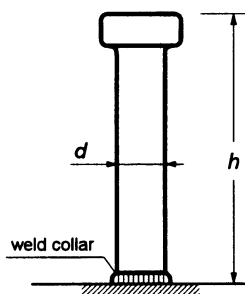


Figure 6.3.1 (*add.*): Welded stud shear connector

6.3.2.2 Influence of tension on shear resistance

mod.

(1) Where headed stud connectors are subjected to direct tensile force in addition to shear, the influence on the design shear resistance should be taken into account. Provided that the design tensile force per stud, $F_{t,Sd}$, does not exceed $0,1 P_{Rd}$, where P_{Rd} is the design shear resistance defined in 6.3.2.1, the influence of the tensile force may be neglected.

(2) Where the force $F_{t,Sd}$ exceeds $0,1 P_{Rd}$, the resistance of the shear connection should be verified by tests.

6.3.3 Headed studs used with profiled steel sheeting

mod.

(1) For the applicability of 6.3.3 of ENV 1994-1-1:1992, see 7.1.1.

6.3.5 Hoops in solid slabs

mod.

(1) Clause 6.3.5 is applicable only to hoops used in combination with block connectors, as shown, for example, in figure 6.7.

(2) The design resistance to longitudinal shear for each leg of a hoop should be determined from

$$P_{Rd} = \frac{A_s f_{yd}}{\sqrt{(1 + \sin^2 \alpha)}}$$

(6.19)

where A_s is the cross-sectional area of one leg of the hoop,
 α is the angle between the planes of the hoop and the flange of the beam,
 f_{yd} is the design strength of the material of the hoop, to be taken as f_y/γ_a or f_{sk}/γ_s whichever is applicable,
 γ_a, γ_s are the partial safety factors for structural steel and reinforcement in accordance with 2.3.3.2.

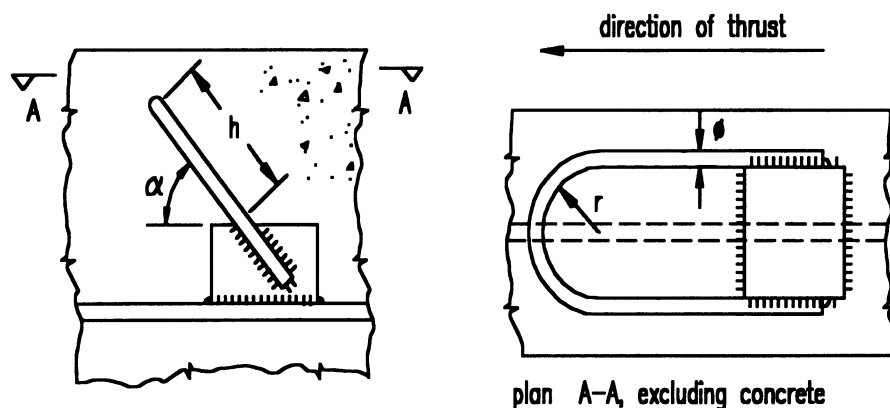


Figure 6.7 (mod.): Example of combination of block connector with hoop

6.3.6 Block connectors with hoops in solid slabs

mod.

(1)P Where a block connector is assumed to share load with a hoop, due account shall be taken of the relative stiffnesses of the block connector and the hoop.

(2) In the absence of more accurate calculations or tests, the design resistance of the combination should be determined from the following expression:

$$P_{Rd,comb} = P_{Rd,block} + 0,7 P_{Rd,hoop}. \quad (6.21)$$

(3) The welds fastening the block connector with hoop to the steel beam should be designed for the forces: $1,2 P_{Rd,block}$ plus $P_{Rd,hoop}$.

6.3.8 Resistance to fatigue of stud connectors in solid slabs

add.

(1) Clause 6.3.8 is applicable only to welded headed stud connectors that satisfy the serviceability condition of 6.1.3(2).

(2) The fatigue strength curve of an automatically welded headed stud with a normal weld collar in concrete with normal-weight aggregate is defined by:

$$\log N = \log a - m \log \Delta \tau_R \quad (6.24)$$

where N is the number of stress range cycles,
 m is the slope constant of the fatigue strength curve, with the value $m = 8$,
 a is a constant which depends on the slope and fatigue strength, with the value $\log_{10} a = 22,123$,
 $\Delta \tau_R$ is the fatigue strength, given by $\Delta \tau_R = 4 \Delta P_R / (\pi d^2)$
 where ΔP_R is the fatigue resistance of one stud and
 d is the nominal diameter of the stud.

The numerical values for the fatigue strength curve are as shown in figure 6.3.8.

(3) For studs in lightweight-aggregate concrete in density classes 1,6 to 2,0 (as defined in 3.1.2.1 of ENV 1992-1-4:1994), 6.3.8(2) is applicable with $\Delta \tau_R$ replaced by $\Delta \tau_{RL}$, given by $\Delta \tau_{RL} = \Delta \tau_R$ (density class) / 2,2,

and $\Delta\tau_c$ replaced by $\Delta\tau_{cL}$, given by $\Delta\tau_{cL} = 95 \text{ (density class)} / 2,2 \text{ N/mm}^2$.

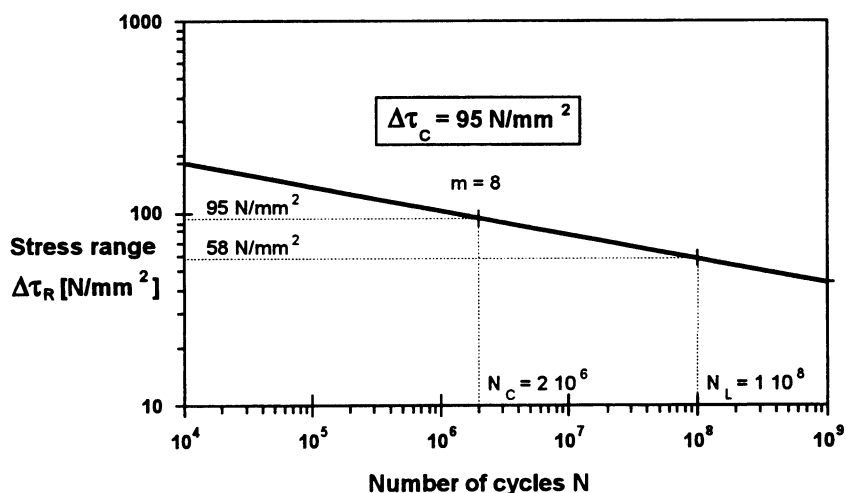


Figure 6.3.8 (add.): Fatigue strength curve for fatigue stress range $\Delta\tau_R$, for headed studs in solid slabs

6.4 Detailing of the shear connection

6.4.1 General recommendations

6.4.1.2 Cover and compaction of concrete

(2) The concrete cover to shear connectors should satisfy the provisions for reinforcing *mod.* steel in 4.1.3.3 of ENV 1992-1-1:1991 and ENV 1992-2:1996.

(3) does not apply.

6.4.1.4 Haunches

(1) Where a concrete haunch is used between the steel girder and the soffit of *mod.* the concrete slab, the sides of the haunch should lie outside a line drawn at 45° from the outside edge of the connector, and its width should be in accordance with figure 6.9 (*mod.*).

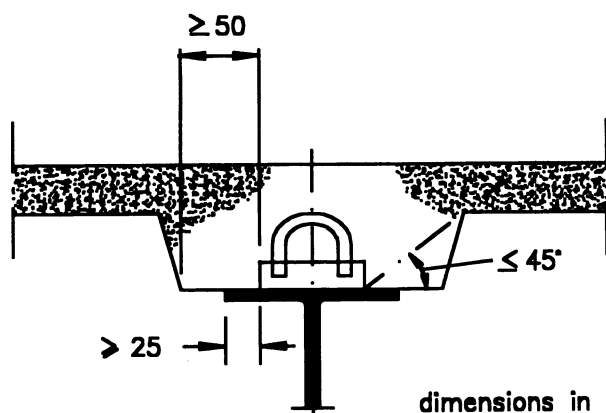


Figure 6.9 (mod.): Dimensions of haunches

6.4.1.5 Spacing of connectors

mod.

(1)P Where it is assumed in design that the stability of either the steel or the concrete element of the member is ensured by the connection between the two, the spacing of the shear connectors shall be sufficiently close for this assumption to be valid.

(2) Where a steel compression flange, that would otherwise be in a lower class, is assumed to be in Class 2 because of restraint from shear connectors within a solid slab:

- the centre-to-centre spacing of the shear connectors in the direction of compression should not exceed

$$25t\sqrt{235/f_y},$$

- the clear distance from the edge of a compression flange to the nearest line of shear connectors should not exceed the lesser of 100 mm and

$$9t\sqrt{235/f_y},$$

- in a composite plate, the centre-to-centre spacing of the shear connectors in the direction perpendicular to the direction of compression should not exceed

$$40t\sqrt{235/f_y}.$$

In these expressions: t is the thickness of the flange, and
 f_y is the nominal yield strength of the flange in N/mm² units.

(3) The maximum longitudinal centre-to-centre spacing of shear connectors should not exceed the lesser of four times the total slab thickness and 800 mm.

(4) Alternatively, headed stud connectors may be placed in groups, with the spacing of groups greater than that specified in (3) for individual connectors, provided that consideration is given in design to:

- the non-uniform flow of longitudinal shear,
- the greater possibility of slip and vertical separation between the slab and the steel member,
- buckling of the steel flange,
- any reduction in the resistance of individual connectors caused by spacing within a group not in accordance with 6.4.2(3),
- the local resistance of the slab to a concentrated force from the connectors.

NOTE: additional detailing rules for connectors for composite plates are given in 7.7.4.

6.4.1.6 Dimensions of the steel flange

(2) The distance between an edge of a connector and the adjacent edge of the flange of the beam to which it is welded should be not less than 25 mm (figure 6.9).
mod.

6.4.2 Stud connectors

(3) The centre-to-centre spacing of studs in the direction of the shear force should be *mod.* not less than $5d$. The spacing in the direction transverse to the shear force should be not less than $4d$ in haunches with sides inclined at more than 30° to the plane of the steel flange, and not less than $2,5d$ otherwise.

(4) Except where the studs are located directly over a steel web, the diameter of a welded *mod.* stud should not exceed 2,5 times the thickness of the flange or plate to which it is welded. For elements in tension and subjected to fatigue loading, this ratio should not exceed 1.5 and should apply also to studs over a web, unless test data are provided to establish the fatigue resistance of the stud as a shear connector.

6.4.3 Headed studs used with profiled steel sheeting *mod.*

(1) For the applicability of 6.4.3 of ENV 1994-1-1:1992, see 7.1.1.

6.4.5 Hoop connectors *mod.*

(1) The anchorage length, concrete cover, and ratio r/ϕ should be in accordance with 5.2.3 of ENV 1992-1-1:1991. The symbols are shown in figure 6.7 (*mod.*).

(2) Hoops designed for longitudinal shear should point in the direction of thrust. Where thrust can occur in both directions, the plane of the hoop should be normal to the plane of the steel flange.

NOTE: These details are shown in figure 6.4 of ENV 1994-1-1:1992

6.5 Friction grip bolts *rep.*

(1)P Where friction grip bolts are used to provide a shear connection between a steel member and a precast concrete slab, their design shall be in accordance with the relevant principles of ENV 1993-1-1:1992 and ENV 1993-2:1997.

(2)P The design shall take account of the reduction of prestressing force in the bolt due to creep and shrinkage of concrete, and of the surface conditions of the steel and concrete at their interface.

(3)P Slip at serviceability limit states shall be limited to a level such that the design satisfies 6.1.3(1)P.

(4)P The design of the connection and the detail at the head of each bolt shall be such that the bearing stress between the steel and the concrete is not excessive.

6.6 Transverse reinforcement

6.6.1 Longitudinal shear in the slab *rep.*

(1)P Transverse reinforcement in the slab shall be designed for the ultimate limit state so that premature longitudinal shear failure and longitudinal splitting are prevented.

(2)P The design longitudinal shear per unit length v_{Sd} for any potential surface of longitudinal shear failure within the slab (figure 6.12) shall not exceed the design resistance to longitudinal shear v_{Rd} of the shear surface considered.

(3) The length of the shear surface b-b shown in figure 6.12 should be taken as equal to $2h$ plus the head diameter for a single row of stud shear connectors or staggered stud connectors, or as equal to $2h + s_t$ plus the head diameter for stud shear connectors arranged in rows, where h is the height of the studs and s_t is the transverse distance centre-to-centre between the studs nearest to the two edges of the flange.

(4) The design longitudinal shear per unit length of beam v_{sd} on a shear surface should be determined in accordance with clause 6.2.

(5) In determining v_{sd} account may be taken of the variation of longitudinal shear across the width of the concrete flange.

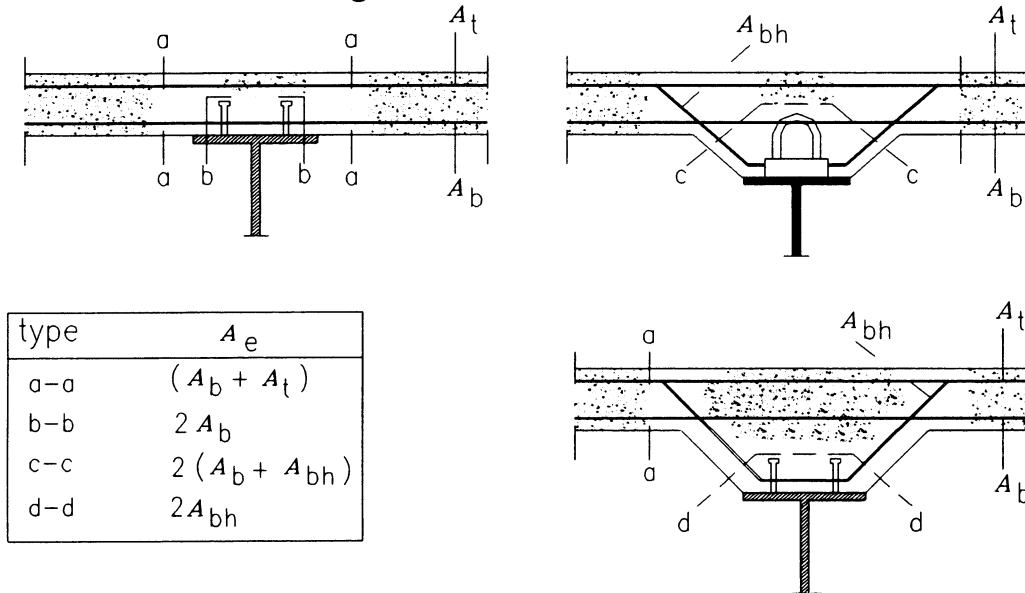


Figure 6.12 (rep.): Typical potential surfaces of shear failure

6.6.2 Design resistance to longitudinal shear

(1) The design resistance of the concrete flange (shear planes a-a illustrated in figure. mod. 6.12) may be determined in accordance with the principles in 4.3.2.5 of ENV 1992-1-1:1991.

(2) In the absence of a more accurate calculation (for example, one that takes account mod. of any transverse prestress), the design resistance of any surface of potential shear failure in a flange or a haunch should be determined from:

$$v_{Rd} = 2,5 A_{cv} \eta \tau_{Rd} + A_e f_{sk} / \gamma_s \quad (6.25)$$

or:
$$v_{Rd} = 0,2 A_{cv} \eta f_{ck} / \gamma_c \quad (6.26)$$

whichever is smaller, where:

- τ_{Rd} is the basic design shear strength, to be taken as $0,25 f_{ctk 0.05} / \gamma_c$, or as zero in a region where the longitudinal shear is determined assuming the concrete to be cracked in longitudinal bending, where $f_{ctk 0.05}$ is defined in 3.1.2.3 of ENV 1992-1-1: 1991;
- f_{ck} is the characteristic cylinder strength of the concrete in N / mm^2 units,
- f_{sk} is the characteristic yield strength of the reinforcement,
- $\eta = 1$ for normal-weight concrete,

$\eta = 0,3 + 0,7 (\rho/24)$ for lightweight-aggregate concrete of unit weight ρ in kN/m³,
 A_{cv} is the mean cross-sectional area per unit length of beam of the concrete shear surface under consideration,
 A_e is the sum of the cross-sectional areas of transverse reinforcement (assumed to be perpendicular to the beam) per unit length of beam crossing the shear surface under consideration (figure 6.12) including any reinforcement provided for bending of the slab,
 A_b and A_{bh} are areas of bottom reinforcement so placed that 6.4.1.1 and 6.4.1.4(3) are satisfied.

6.6.3 Contribution of profiled steel sheeting

mod.

- (1) For the applicability of 6.6.3 of ENV 1994-1-1:1992, see 7.1.1.

6.6.4 Minimum transverse reinforcement in cast in situ solid slabs

rep.

- (1) Minimum reinforcement should be in accordance with 5.4.3 of ENV 1992-1-1:1991.

6.6.5 Longitudinal splitting

rep.

NOTE: The following rules supplement 6.1.4(2)P, and 5.4.3.2.4 of ENV 1992-1-1:1991.

- (1) Where, in a composite beam, the distance from the edge of the concrete flange to the centreline of the nearest row of shear connectors is less than 300 mm:

- the bottom transverse reinforcement in accordance with 6.4.1.1 and 6.6.2 should consist of U-bars passing around the shear connectors, and should be placed as low as possible in the slab;

- where headed studs are used as shear connectors, the distance from the edge of the concrete flange to the centre of the nearest stud should not be less than $6d$, where d is the nominal diameter of the stud, and the U-bars should be not less than $0,5d$ in diameter.

7 Composite slabs with profiled steel sheeting and composite plates

7.1 General

7.1.1 Scope

(5) No details are given about the use of composite slabs with profiled sheeting as a bridge *add.* deck or the composite action of such a slab with the supporting steel members. Unless specified otherwise in the project specification or by the relevant authority the applicability of clauses 7.1 to 7.6 as well as 3.4, 4.2.1(4), 6.3.3, 6.4.3, 6.6.3, 9.4.4, 9.4.5, 10.3 and annex E should be considered by the designer. For the design of non-participating formwork of profiled sheeting, see ENV 1993-1-3:1996.

7.7 Composite plates

add.

7.7.1 General

(1)P This clause 7.7 is valid for composite plates consisting of a nominally flat plate of structural steel connected to a site cast concrete slab by headed studs. Double skin plates or other types of connectors are not covered. The intended use of the composite plate is as a flange in a bridge deck carrying transverse loads as well as in-plane forces, or as a bottom flange in a box girder.

(2) The steel plate should be supported during casting either permanently or by temporary supports in order to limit its deflection to less than 0,05 times the slab thickness unless the additional weight of concrete is taken into account.

(3) When determining the effective width according to 4.2.2.2, b_0 should be taken as $2a_w$ with a_w as defined in 7.7.4(5).

(4) For global analysis, clause 4.5 applies.

7.7.2 Design for local effects

(1) Local effects are bending moments and shears caused by transverse loads on the plate acting as an one- or two-way slab. For the purpose of analysis of local action effects the slab may be assumed elastic and uncracked. A top flange of an I-girder need not be designed as composite in the transverse direction.

(2) The concrete and the steel plate may be assumed to act compositely without slip. The resistance should be verified for bending and vertical shear as for a reinforced concrete slab where the steel plate is considered as reinforcement.

(3) The design resistance for shear in 4.3.2.3 and 4.3.4.5.1 of ENV 1992-1-1:1991 are applicable and the plate may be considered as reinforcement if 7.7.4(4) is satisfied.

7.7.3 Design for global effects

- (1)P The composite plate shall be designed to resist all forces from axial loads and global bending of all longitudinal girders or cross-girders that it forms part of.
- (2) The design resistance for compression may be taken as the sum of the design resistance of the concrete and the steel plate within the effective width according to 4.2.2 provided that the steel plate is connected to the concrete according to clause 6.4. Reduction in strength due to buckling should be considered according to ENV 1992-1-1:1991, if appropriate.
- (3) The design resistance for tension should be taken as the sum of the design resistance of the steel plate and the reinforcement within the effective width according to 4.2.2
- (4) Interaction with local load effects should be considered for the shear connectors as stated in 7.7.4 (1)P. Otherwise it need not be considered.

7.7.4 Design of shear connectors

- (1)P Resistance to fatigue and requirements for serviceability limit states shall be verified for the combined local and simultaneous global effect. Connectors that carry loads both longitudinally and transversely may be verified for the vector sum of the simultaneous forces on the connector.
- (2) The design resistance of stud connectors in 6.3.2 and 6.3.8 may be used provided that the concrete slab has bottom reinforcement with area not less than 0,002 times the concrete area in each of two perpendicular directions according to 6.4.1.1(1).
- (3) The detailing rules of clause 6.4 are applicable.
- (4) In order to justify the use of the steel plate as reinforcement (see 7.7.2(2)) the distance, in two perpendicular directions, between connectors should not exceed three times the slab thickness.
- (5) For wide girder flanges the distribution of longitudinal shear due to global effects for serviceability limit states may be determined as follows in order to account for slip and shear lag. The longitudinal force P_{sd} on a connector at distance x from the nearest web may be taken as

$$P_{sd} = \frac{v_{sd}}{n} \left[\left(3,85 \left(\frac{n_w}{n} \right)^{-0,17} - 3 \right) \left(1 - \frac{x}{b} \right)^2 + 0,15 \right] \quad (7.10)$$

where

v_{sd} is the design longitudinal shear flow due to global effects,

- n is the total number of connectors of the same size per unit length of girder within the width b in Fig. 7.13, provided that the number of connectors per unit area does not increase with x ,
- n_w is the number of connectors per unit length placed within a distance from the web a_w equal to the larger of $10t_f$ and 200 mm,
- b is equal to half the distance between adjacent webs or the distance between the web and the free edge of the flange.

In case of a flange projecting up to a_w outside the web, n and n_w may include the connectors placed on the flange. For efficiency, n_w should be as high as possible and the spacing of the connectors should be small enough to avoid premature local buckling of the plate, see 6.4.1.5(2).

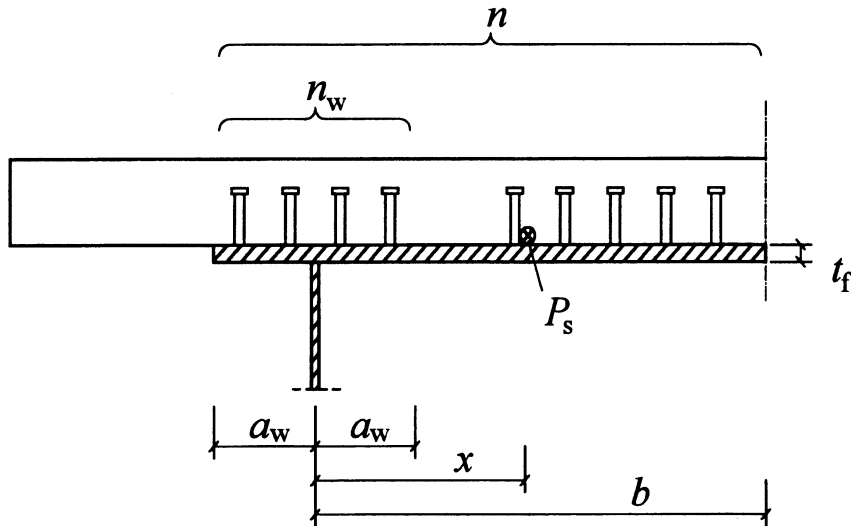


Figure 7.13: Definition of notations in equation (7.10)

8 Decks with precast concrete slabs

8.1 General

(1)P This Section 8 deals with reinforced or prestressed precast concrete slabs or planks, *mod.* used either as full depth flanges of bridge decks or as partial depth slabs acting with insitu concrete.

(2)P The precast bridge slab shall be designed in accordance with 5.4.9 of *mod.* ENV 1992-2:1996 and also for composite action with the steel beam.

(3) Relevant tolerances should be given in the project specification.

8.2 Actions

(3) Clauses 7.3.2.1(2) and 7.3.2.1(4) are applicable to precast elements acting as *mod.* permanent formwork. These minimum loads are not necessarily sufficient and the requirements of the construction method should be taken into account.

8.3 Partial safety factors for materials

(1)P For the structural steel, reinforcement and concrete the safety factors given in 2.3.3 *mod.* and 2.3.4 shall be used.

(2) Does not apply

8.4 Design, analysis and detailing of the bridge slab

rep.

(1) The precast slab together with any insitu concrete should be designed as continuous in both the longitudinal and the transverse directions. The joints between slabs should be designed to transmit membrane forces as well as bending moments and shears. Compression perpendicular to the joint may be assumed to be transmitted by contact pressure if the joint is filled with mortar or glue or if it is shown by tests that the mating surfaces are in sufficiently close contact.

(2) For the use of stud connectors in groups, see 6.4.1.5(4). For connections with friction grip bolts, see clause 6.5. The friction coefficient between slab and steel beam should be established by tests.

(3) The stepped distribution of shear forces in 4.5.3.2(104) of ENV 1992-1-3:1994 may be used provided that the limitations in 6.1.3(2) are observed.

8.5 Joints between steel beam and concrete slab

8.5.1 Bedding and tolerances

(1)P Where precast slabs are supported on steel beams without bedding the influence of the *mod.* vertical tolerances of the bearing surfaces shall be considered.

8.5.2 Corrosion

- (1) A steel flange under precast slabs without bedding should have the same corrosion *mod.* protection as the rest of the steelwork but for a top coating provided after erection. Bedding with the purpose of protecting against corrosion may be designed to be not loadbearing.
- (2) Does not apply.

8.5.3 Shear connection and transverse reinforcement *mod.*

- (1) The shear connection and transverse reinforcement should be designed in accordance with the relevant clauses of Section 6.
- (2) If shear connectors welded to the steel beam project into recesses within slabs or joints between slabs, which are filled with concrete after erection, the detailing and the properties of the concrete should be such that it can be cast properly.
- (3) In absence of relevant experience the minimum thickness of infill around such shear connectors should be at least 25mm.
- (4) If shear connectors are arranged in groups, sufficient reinforcement should be provided near each group to prevent premature local failure in either the precast or the insitu concrete.
- (5) Does not apply.

8.6 Concrete floor designed for horizontal loading Does not apply.

9. Execution

9.2 Sequence of construction

(3) The rate and sequence of concreting should be specified to be such that partly *mod.* matured concrete is not damaged as a result of limited composite action occurring from deformation of the steel beams under subsequent concreting operations. Wherever possible, deformation should not be imposed on a shear connection until the concrete has reached a cylinder strength of at least 20 N/mm².

9.4 Accuracy during construction, and quality control

9.4.1 Deflection during and after concreting

(1)P Clause 5.4 is applicable.
mod.

(2) The formwork and its supports should be capable of transferring the load and effects *rep.* of concreting to the steelwork or other supports without distress and in accordance with the design assumptions.

(3) does not apply.

9.4.3 Shear connection

9.4.3.1 Headed studs

rep.

(1)P Arc stud welding shall be in accordance with prEN ISO 14555 'Welding-Arc stud welding of metallic materials' and prEN ISO 13918 'Welding-Studs for arc stud welding'.

9.4.3.2 Hoops and block connectors

rep.

(1)P The welding of hoops and block connectors shall be in accordance with the relevant clauses of ENV 1992-1-1:1991 and ENV 1993-1-1:1992.

(2)P Hoops that shall be welded shall comply with the conditions of weldability given in ENV 1992-1-1:1992.

9.4.3.3 Friction grip bolts

rep.

(1)P The interface between the steel member and the concrete flange shall be free of oil, dirt, rust, loose mill scale, burrs and other defects which would prevent a uniform sealing between the two elements, or would interfere with the development of friction between them.

(2) The method should be in accordance with the relevant clauses of Section 7 of ENV 1993-1-1:1992.

9.4.3.4 Corrosion protection in the interface*rep.*

- (1) Does not apply.
- (2) Corrosion protection as applied to the steelwork should extend at least 50 mm into the interface, except that any final site coat may normally be omitted.

9.4.3.5 Surface condition*mod.*

- (1) For composite columns and filler beam decks, the surface of the steel section, in contact with the concrete filling or encasement should be unpainted and free from oil, grease and loose scale or rust.

9.4.4 Composite slabs with profiled steel sheeting

- (1) For the applicability of 9.4.4 of ENV 1994-1-1:1992, see 7.1.1

10 Design assisted by testing

10.1 General

(1)P Unless otherwise specified, Section 8 of ENV 1993-1-1:1992 applies.

mod.

(2) Specific additional rules are given in clause 10.2 for static tests on shear connectors.

mod. Specimens of the types specified in 10.2.1 to 10.2.3 may also be used for the fatigue testing of shear connectors.

add. NOTE: General information on testing procedure, evaluation of results, and the determination of design resistance from tests is given in annex D of ENV 1991-1:1994 and annex Z of ENV 1993-1-1/A2:1997.

10.3 Testing of composite floor slabs

mod.

(1) For the applicability of clause 10.3 of ENV 1994-1-1:1992, see 7.1.1(5).

Annex K (normative)

Filler beam decks

K.1 General

(1)P In this annex, clauses K.2 to K.6 concern decks consisting of a concrete slab reinforced by longitudinal steel filler beams and by reinforcing steel.

(2) A typical cross-section of filler beam deck is shown in figure K.1. No application rules are given for fully encased beams.

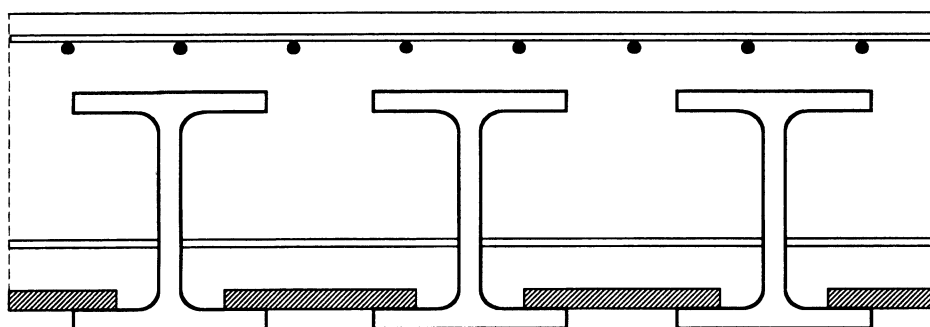


Figure K.1 : Typical cross-section of a filler beam deck

(3) Clause K.7 is applicable to half-through bridges where the deck consists of a concrete slab reinforced by transverse steel filler beams and by reinforcing steel. A typical cross-section of a half-through bridge is shown in figure K.2.

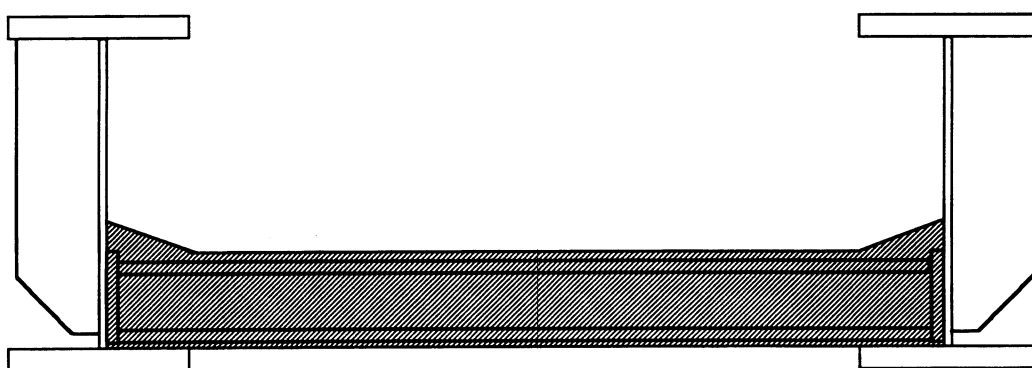


Figure K.2 : Typical cross-section of a half-through bridge

(4) In this annex, "longitudinal" means the direction of the longitudinal axis of the bridge, and "transverse" is relative to this longitudinal axis.

(5)P Filler beam decks shall be designed for the serviceability and ultimate limit states. Moreover, steel beams that incorporate welding shall be checked against fatigue.

K.2 Requirements

(1)P Spans may be simply supported or continuous. Supports may be skew or not. Provided that the provisions in this annex are met, then mechanical shear connection need not be provided.

(2) Steel beams may be rolled sections, or welded sections with a constant cross-section. For welded sections, both the width of the flanges and the depth of the web should be within the ranges that are available for rolled H- or I- sections.

(3) Decks with longitudinal filler beams, that are within the scope of this annex, should comply with all of the following conditions :

- steel beams are straight in horizontal projection ;
- the skew θ of all the supports complies with : $0 \leq \theta \leq 30^\circ$ (the value $\theta = 0$ corresponding to a non-skew deck) ;
- the nominal depth h of the steel beams complies with : $0,21 \text{ m} \leq h \leq 1,10 \text{ m}$;
- the spacing of webs of the steel beams does not exceed the lesser of $h/3 + 0,60 \text{ m}$ and $0,75 \text{ m}$, h being the nominal depth of the steel beams in metres ;
- the concrete cover c above the steel beams complies with the three following conditions :

$$c \geq 70 \text{ mm}$$

$$c \leq 150 \text{ mm}$$

$$c \leq h/3$$

- the soffit of the lower flange of the steel beams is not encased ;
- a bottom layer of transverse reinforcement passes through the webs of the steel beams, and is anchored beyond the end steel beams, and at each end of each bar, so as to develop its yield strength in accordance with ENV 1992-2 : 1996 ; high bond bars are used ; their diameter is not less than 16 mm and their spacing is not more than 300 mm ;
- normal-weight concrete is used.

K.3 Global analysis

(1) This clause K.3 is applicable for checking all the limit states, including fatigue.

(2) Internal forces and moments should be determined from uncracked elastic analysis.

(3) For spans of continuous filler beams with class 1 cross-sections, the elastic bending moments for ultimate limit states may be redistributed to take account of inelastic behaviour of materials. This redistribution may be carried out by reducing hogging moments by amounts not exceeding 15% of the initial value of the bending moment to be reduced. For each load case, the internal forces and moments after redistribution should be in equilibrium with the loads.

(4) For the calculation of deflections in serviceability limit states, the constant value I of the second moment of area of cross-sections of filler beam decks may be taken as :

$$I = \frac{I_1 + I_2}{2}$$

where I_1 and I_2 are the "uncracked" and the "cracked" values of the second moment of area of the composite cross-section subjected to sagging bending.

(5)P Where the distribution of the loads applied after hardening of the concrete is not uniform in the direction transverse to the span of the filler beams, the analysis shall take account of any difference between the deformations of adjacent filler beams, unless it is verified that a sufficient accuracy is obtained by a simplified analysis assuming rigid cross-sections.

(6) Account may be taken of these deformations by using one of the following methods of analysis :

- smearing of the steel beams so as to have an orthotropic continuum ;
- considering the concrete as discontinuous so as to have a plane grid with members having flexural and torsional stiffness ;
- general methods, such as the use of finite elements of suitable shape.

The torsional stiffness of steel beams may be neglected.

(7) The nominal value of Poisson's ratio, if needed for calculations, may be assumed to be, in all directions :

- 0 for the verifications of ultimate limit states ;
- 0,2 for the verifications of serviceability limit states.

(8) Unless otherwise specified, account may be taken of the deformations of concrete due to creep by the method of modular ratios, and two nominal values of the "effective" modulus of concrete E'_c may be adopted : one equal to E_{cm} for short term effects, the other equal to $\frac{E_{cm}}{3}$ for long term effects.

3

(9) The influence of shrinkage of concrete may be ignored.

(10) The influence of differences and gradients of temperature may be ignored, except for the deflections calculation of non-ballasted railway bridges.

(11) In longitudinal bending, the effects of slip between the concrete and the steel beams may be ignored.

(12) In transverse bending, the presence of steel beams may be ignored.

(13) The influence of shear lag may be ignored.

(14) The contribution of formwork supported from the steel beams, which becomes part of the permanent construction, as shown on figure K.1, should be neglected.

K.4 Ultimate limit states

K.4.1 General

- (1)P Composite cross-sections shall be classified according to Section 4 with the following change : webs may be considered as encased.
- (2) The encasement of webs has two consequences :
 -a web in class 3 that is encased may be represented by an effective web of the same cross-section in class 2 ;
 -the classification of outstand steel flanges should be in accordance with Table K.1, in which ϵ is defined in Table 4.2.

**Table K.1 : Maximum width-to-thickness ratios
for outstand steel flanges in compression**

class	type of section	maximum width-to-thickness ratios
1	Rolled Welded	$c/t \leq 10\epsilon$ $c/t \leq 9\epsilon$
2	Rolled Welded	$c/t \leq 15\epsilon$ $c/t \leq 14\epsilon$
3	Rolled Welded	$c/t \leq 21\epsilon$ $c/t \leq 20\epsilon$

- (3) Application rules K.3 (8) to K.3 (14) are applicable for checking cross-sections for ultimate limit states.

K.4.2 Bending moments

- (1)P The design resistance of composite cross-sections to bending moments shall be determined according to Section 4.
- (2)P The design resistance of reinforced concrete sections to bending moments shall be determined according to ENV 1992-2:1996.

K.4.3 Vertical shear

- (1)P The resistance of composite cross-sections to vertical shear shall be taken as the resistance of the structural steel section alone, unless the value of a contribution from the reinforced concrete part has been established and verified according to ENV 1992-2:1996.
- (2)P The design resistance to vertical shear of reinforced concrete sections between filler beams shall be verified according to ENV 1992-2 : 1996.

K.4.4 Strength and stability of steel beams during construction

(1)P Steel beams before the hardening of concrete shall be verified. The calculations shall be carried out according to ENV 1993-2:1997.

K.5 Serviceability limit states

K.5.1 General

- (1)P The serviceability limit states shall be checked according to :
- ENV 1992-2:1996 for checks related to reinforced concrete cross-sections ;
 - Section 5 for other checks.
- (2) Application rules K.3 (8) to K.3 (14) are applicable for checking cross-sections for serviceability limit states.

K.5.2 Cracking of concrete

- (1)P The principles in 5.3.1 are applicable.
- (2) The application rules in 5.3.1 are applicable.
- (3) K.5.3 and K.5.4 below apply to the reinforcing bars or wires in the direction of the steel beams within the whole thickness of the deck.

K.5.3 Minimum reinforcement

- (1) The minimum reinforcement should be determined in accordance with 4.4.2.2.3 (101) of ENV 1992-2:1996, using a value $k_c = 0,4$.

K.5.4 Control of cracking

- (1) 4.4.2.3 of ENV 1992-2:1996 applies. The stresses in the reinforcement should be calculated by using the cross-section properties of the cracked composite section with the second moment of area I_2 according to 4.2.3 (2).

K.6 Detailing

- (1) The clear distance between the upper flanges of the steel beams should not be less than 0,15 m so as to allow pouring of concrete.
- (2) The minimum concrete cover for the flanges of the steel beams at the sides of the deck should not be less than 80 mm.
- (3) The surface of the steel beams should be de-scaled.
- (4)P The soffit, the upper surfaces and the edges of the lower flange of the steel beams shall be protected against corrosion.

K.7 Half-through bridges with transverse filler beams

K.7.1 General

- (1)P The scope of clause K.7 is transverse filler-beam decks in which either:
-shear connection is provided in accordance with Section 6, or
-the conditions of K.2(3) are applicable, or are modified to an extent supported by relevant experience of satisfactory long-term behaviour of transverse filler-beam decks subjected to loading similar to that specified for the particular project.
- (2)P The principles of clause K.1 and of clauses K.3 to K.6 are applicable.
- (3)P No application rules are given for transverse filler-beam decks.

K.7.2 Analysis

- (1)P Spans of transverse filler beams may be designed as simply supported. Where transverse filler beams act as part of transverse U-frames which stabilise main beams, the appropriate action effects shall be considered.

K.7.3 Shear in the direction of span of the transverse beams

- (1)P This shear shall be determined by elastic theory, taking account of the possibilities that :
- global longitudinal tension and/or the effects of shrinkage or temperature may have caused the concrete to be cracked, or
- the tensile strength of the concrete may be higher than specified, and that it is uncracked.
- (2)P In determining which interfaces between steel and concrete are assumed to resist longitudinal shear by bond and/or friction, account shall be taken of any action effects that tend to cause separation at interfaces, including :
- global longitudinal tension in the slab,
- effects of shrinkage and cracking of concrete.
- (3)P For all assumed transfers of this shear across interfaces between structural steel and concrete, appropriate routes and effective areas shall be assumed, using one or more of the following:
- shear connection in accordance with Section 6,
- shear or direct compression across surfaces where the coexisting action effects do not tend to cause separation
- dowel action of reinforcing bars that pass through holes in structural steel members.
If more than one method of shear transfer is used, it shall be ensured that the load-slip behaviours of the methods are compatible. Assumptions shall be based on testing or on extensive experience of long term behaviour under repeated loading.
- (4)P The values of any coefficient of friction or any bond strength used in design shall be based on testing or on extensive experience of long term behaviour under repeated loading.

K.7.4 Detailing

(1)P The detailing shall be designed to ensure that no rainwater can penetrate between the web of a longitudinal beam and the waterproofing layer of the deck.

Annex L (informative)

Effects of tension stiffening in composite bridges

L.1 Scope

(1) In this annex a method is given to treat the effect of tension stiffening in the decks of composite road and railway bridges. The annex is applicable for bridges with or without prestressing by tendons or imposed deformation.

(2) The deck can act as a tension member in composite systems such as in bowstring arches and trusses according to 4.7.1, or in the tension regions of a composite beam. Clauses L.3 to L.5 are applicable to composite cross-sections where the concrete slab is above the steel beam.

L.2 Tension members in bowstring arches and trusses

(1) Unless a more precise method is used, the effect of tension stiffening according to (2) may be taken into account by use of the average normal force-strain curve A for the reinforced concrete element given in figure L.1.

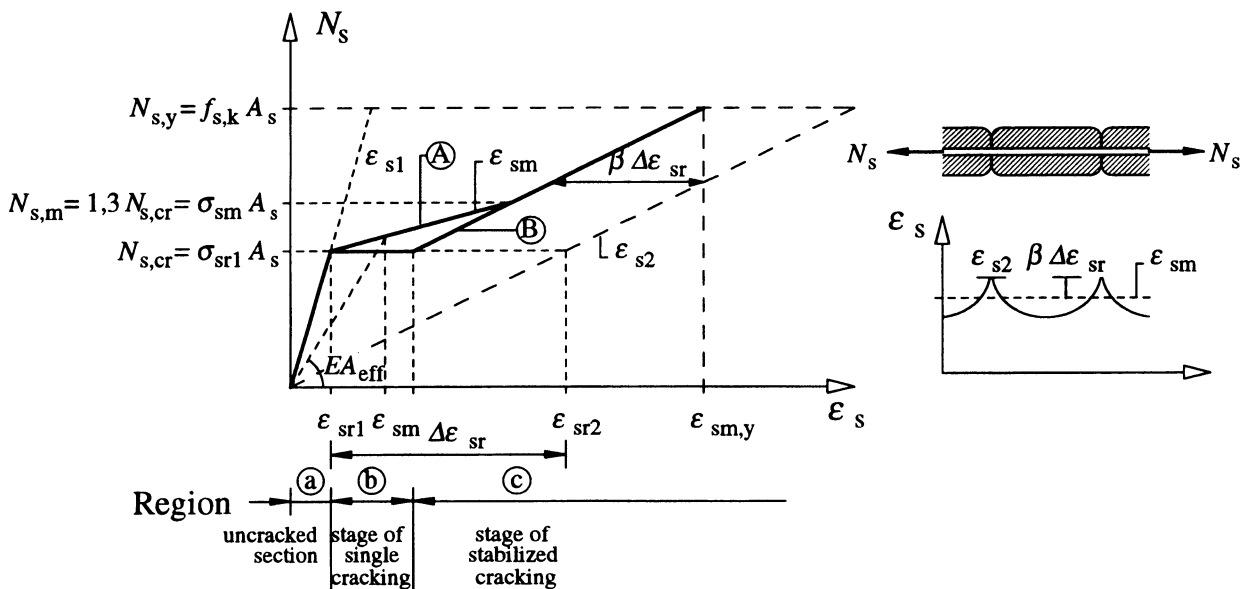


Figure L.1: Normal force and mean strain for reinforced concrete tension members

(2) For verification the three regions (a), (b) and (c) according to the actual reinforcement stress can be distinguished. Region (a) gives the behaviour of the uncracked section, region (b) the behaviour in the stage of initial crack formation and region (c) the behaviour in the stage of stabilised crack formation.

(a) Uncracked ($0 \leq N_s \leq N_{s,cr}$)

$$\varepsilon_{sm} = \varepsilon_{s1} \quad (L.1)$$

(b) Initial crack formation phase ($N_{s,cr} < N_s \leq N_{s,m}$)

$$\varepsilon_{sm} = \varepsilon_{s2} - \frac{\beta (N_s - N_{s,cr}) + (N_{s,m} - N_s)}{(N_{s,m} - N_{s,cr})} (\varepsilon_{sr2} - \varepsilon_{sr1}) \quad (L.2)$$

(c) Stabilised cracking ($N_{s,m} < N_s \leq N_{s,y}$)

$$\varepsilon_{sm} = \varepsilon_{s2} - \beta (\varepsilon_{sr2} - \varepsilon_{sr1}) = \varepsilon_{sr2} - \beta \Delta \varepsilon_{sr} \quad (L.3)$$

where:

ε_{s1} is the reinforcement strain in the uncracked stage,

ε_{s2} is the reinforcement strain in the cracked stage neglecting the effect of tension stiffening of concrete between cracks,

ε_{sr1} is the reinforcement strain in the uncracked stage under cracking forces reaching the mean value f_{ctm} of concrete tensile strength according to 3.1.2 (3). In case of tension members with additional local bending moments due to permanent loads and traffic loads (e.g. decks in tied arch bridges) that cause stresses in the same direction as the normal force instead of the tensile strength f_{ctm} the value $0,7 f_{ctm}$ may be used,

ε_{sr2} is the reinforcement strain at the crack under cracking forces reaching f_{ctm} (if the internal forces are lower than or equal to the cracking forces, then $\varepsilon_{sr2} = \varepsilon_{s1}$),

σ_{sr1} is the reinforcement stress in the crack, when the first crack has formed,

σ_{sm} is the reinforcement stress in the crack, when the stabilized crack pattern has formed. If no further information is given, $\sigma_{sm} = 1,3 \sigma_{sr1}$, may be used,

$N_{s,cr}$ is the force in the reinforcement in the crack, when first crack has formed ($N_{s,cr} = \sigma_{sr1} A_s$),

$N_{s,m}$ is the force in the reinforcement in the crack, when the stabilized crack pattern has formed. If no further information is given, $N_{s,m} = 1,3 N_{s,cr}$ may be used,

$\beta = 0,40$ for deformed bars,

f_{sk} is the characteristic yield strength of reinforcement according to clause 3.2.

(3) For the determination of bending moments in composite tension members according to clause 4.7 caused by deformations of the main structure (not by local vertical loads), the flexural stiffness of the cracked tension member may be calculated by multiplying the flexural stiffness of the uncracked reinforced concrete element by the ratio of its longitudinal stiffness $E A_{\text{eff}}$ with the stiffness $E A$ of the uncracked section, where the longitudinal stiffness $E A_{\text{eff}}$ is given as $E A_{\text{eff}} = N_s / \varepsilon_{\text{sm}}$, where ε_{sm} is shown in figure L.1.

L.3 Tension members in composite beams

(1) In figure L.2 a cross-section of a composite beam is shown, where the bending moment causes tension in the concrete slab. The moment M can be replaced by the forces N_a and M_a and N_s and M_s on the individual sections, where the bending moment M_s in the tension member may be neglected ($M_s \approx 0$) for the determination of N_s . The moment $M = M_a + N_s a$ in the cross-section is distributed to a moment of the steel section M_a and to normal forces $N_a = -N_s$ acting at the lever arm a .

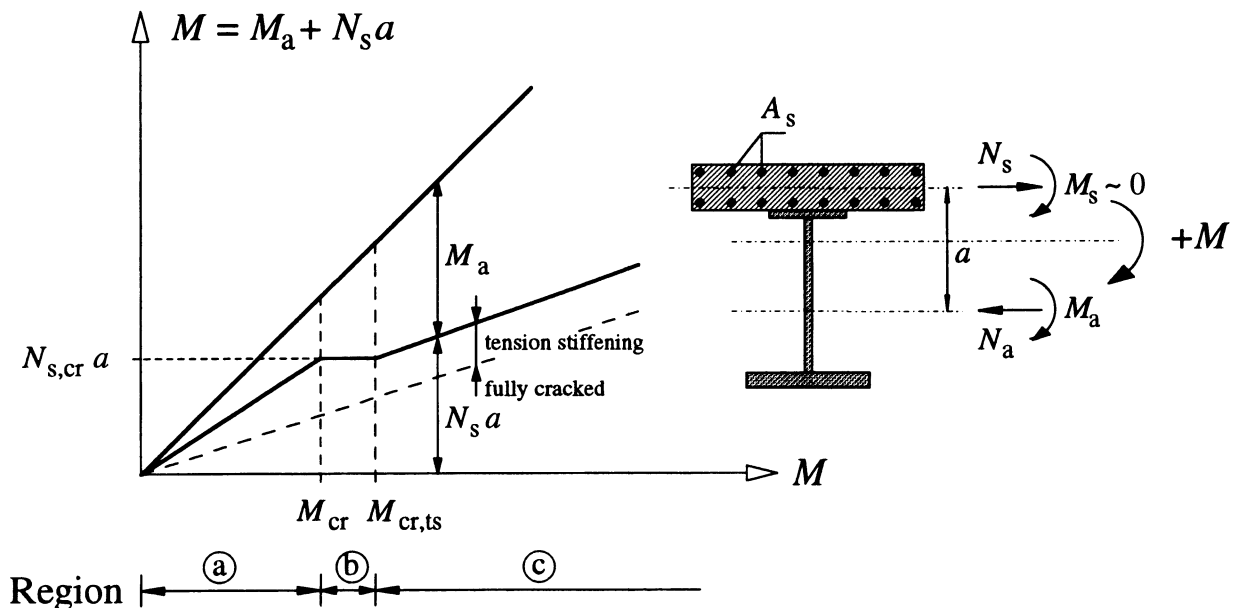


Figure L.2: Contribution of internal forces to total bending moment M without shrinkage

(2) Unless a more precise method is used the effect of tension stiffening on N_s may be taken into account by use of the average force-strain curve (B) given in figure L.1.

(3) The cracking moment M_{cr} is the moment at the beginning of the initial crack formation phase. $M_{\text{cr,ts}}$ is the moment at the beginning of the stabilized crack formation.

(4) The internal forces on the individual cross-sections, not prestressed by tendons, prior to cracking resulting from the isostatic effects of shrinkage and from the cracking moment M_{cr} are shown in figure L.3. The isostatic effects of shrinkage may be determined according to 4.2.3 (4).

(5) For cross sections without prestressing by tendons the cracking moment M_{cr} is defined as the bending moment that causes, in addition to the isostatic effects of shrinkage, the stress at the extreme fibre of the concrete section to reach the mean tensile concrete strength f_{ctm} . The internal forces and stresses due to M_{cr} should be determined using the modular ratio n_0 for short time loading according to 3.1.4.2. For the definition of M_{cr} for sections prestressed by tendons, the isostatic effect of prestress should be included.

(6) For cross sections without prestressing by tendons the internal normal force of the concrete section caused by shrinkage and the cracking moment M_{cr} is defined as the internal force $N_{s,cr}$ (see figures L.3 and L.4). For sections prestressed by tendons, the internal normal force $N_{s,cr}$ should be calculated including the isostatic effect of prestress.

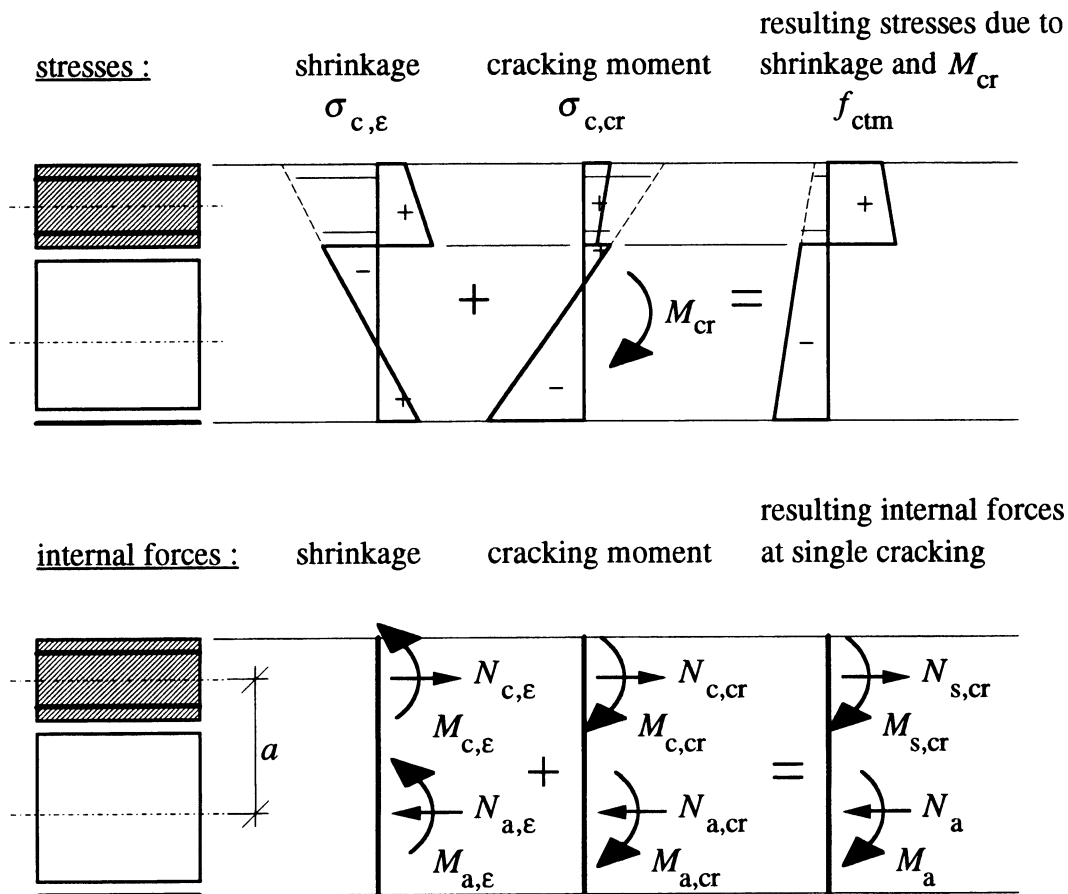


Figure L.3 Definition of the cracking moment M_{cr} , region (a), taking into account shrinkage

(7) It may be assumed, that single cracking governs for moments not greater than $M_{cr,ts}$. It results from the condition (figure L.4) that the internal normal force of the concrete section $N_s = N_{s0} + \Delta N_{s,ts}$ according to (10) is equal to $N_{s,cr}$.

$$M_{cr,ts} = (N_{s,cr} - \ddot{A}N_{s,ts}) \frac{I_2}{A_s z_2} \quad (L.4)$$

where:

I_2 is the second moment of area of the composite section neglecting concrete according to 4.2.3 (2),

z_2 is the distance between the centroidal axes of the cracked composite section with the second moment of area I_2 and the centre of area of the reinforcement,

$\Delta N_{s,ts}$ is defined in (10).

(8) In the stage of single cracking it may be assumed that the internal normal force of the concrete chord $N_{s,cr}$ due to shrinkage and a bending moment M greater than M_{cr} is constant.

(9) In the stage of single cracking the reduction of isostatic effects due to shrinkage caused by cracking may be taken into account by assuming a linear reduction of the force $N_{s,\epsilon}$ and the bending moment $M_{s,\epsilon}$ (see figure L.4) between the corresponding values of the bending moment M_{cr} and the bending moment $M_{cr,ts}$ at the beginning of stabilized cracking where $N_{s,\epsilon}$ and $M_{s,\epsilon}$ may be assumed to be zero.

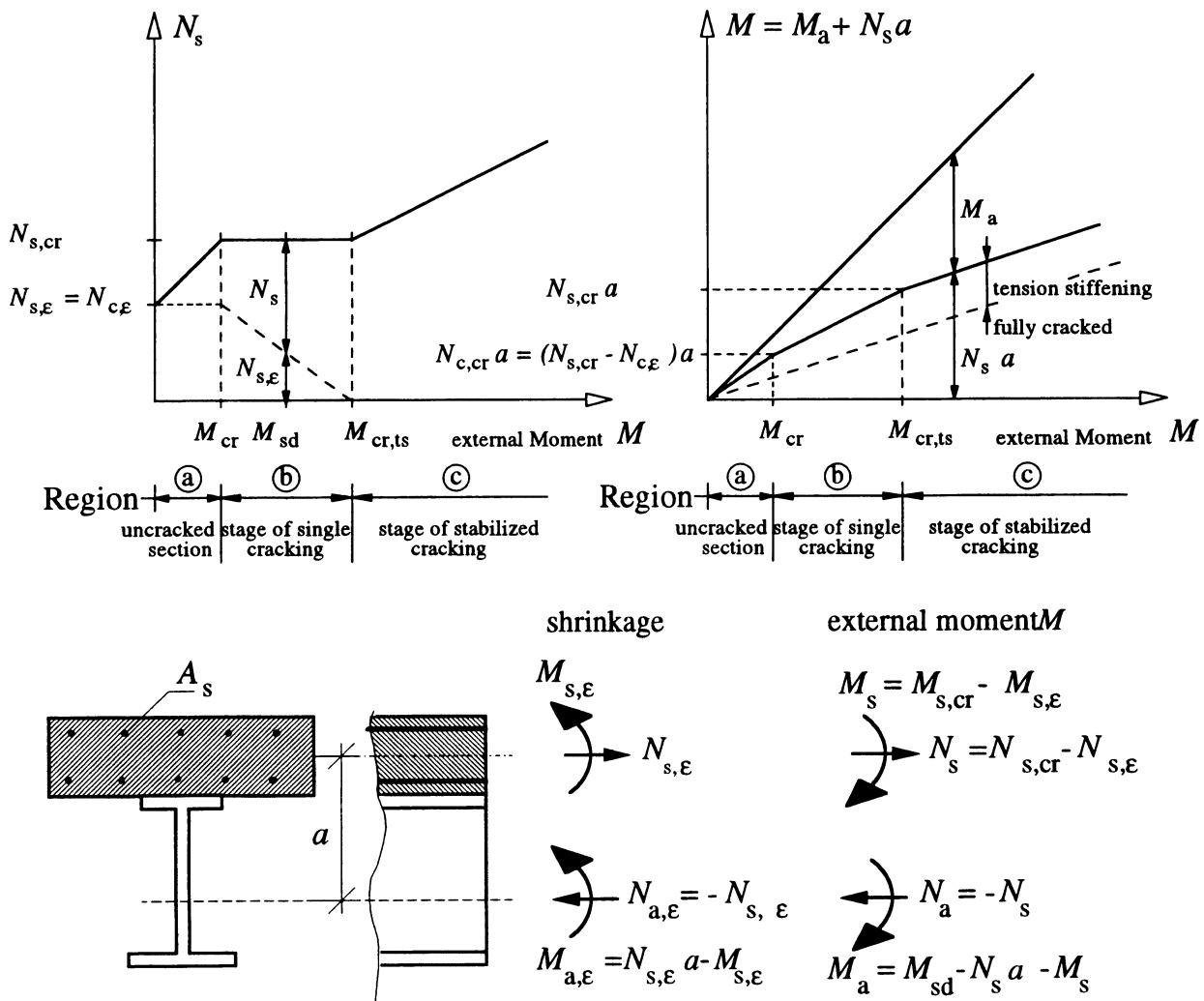


Figure L.4: Internal forces of the individual cross-sections for single cracking, region (b), taking into account shrinkage

(10) In the stage of stabilized cracking the internal forces of the sections result from adding the internal cross-sectional forces due to M_{sd} determined with the composite section neglecting concrete (N_{s0} , M_{s0} , M_{a0} , N_{a0}) and the additional internal forces $\Delta N_{s,ts}$, $\Delta N_{a,ts}$ and $\Delta M_{a,ts}$ due to tension stiffening, according to figure L.5.

$$N_s = N_{s0} + \Delta N_{s,ts} \quad \Delta N_{a,ts} = -\Delta N_{s,ts} \quad \Delta M_{a,ts} = \Delta N_{s,ts} a \quad (L.5)$$

The additional normal force $\Delta N_{s,ts}$ of the concrete section due to tension stiffening for bridges without prestressing by tendons is given by:

$$\Delta N_{s,ts} = 0,4 \frac{f_{ctm} A_s}{\rho_s \alpha_{st}} \quad (L.6)$$

The additional normal force $\Delta N_{s,ts}$ of the concrete section due to tension stiffening for bridges with prestressing by tendons is given by:

$$\Delta N_{s,ts} = 0,4 \frac{f_{ctm} (A_s + A_p)}{eff \rho_p \alpha_{st}} \quad (L.7)$$

where:

f_{ctm} is the mean tensile strength of concrete according to Section 3,

A_s are the areas of reinforcement steel within the effective width,

A_p are the areas of prestressing steel within the effective width,

a is the distance between the neutral axes of the structural steel section and the reinforced and uncracked concrete section,

$\alpha_{st} = A I_2 / (A_a I_a)$, where A and I_2 are the area and the second moment of area, respectively, of the composite section neglecting concrete in tension; and A_a and I_a are the corresponding properties of the structural steel section,

A_{ct} is the area of the tensile zone immediately prior to cracking of the cross section, for simplicity the area of the concrete section within the effective width should be used,

$\rho_s = (A_s / A_{ct})$,

$eff \rho_p$ is the effective reinforcement ratio according to 5.3.3.2 (2).

(11) In the stage of stabilized cracking the primary effects of shrinkage may be neglected.

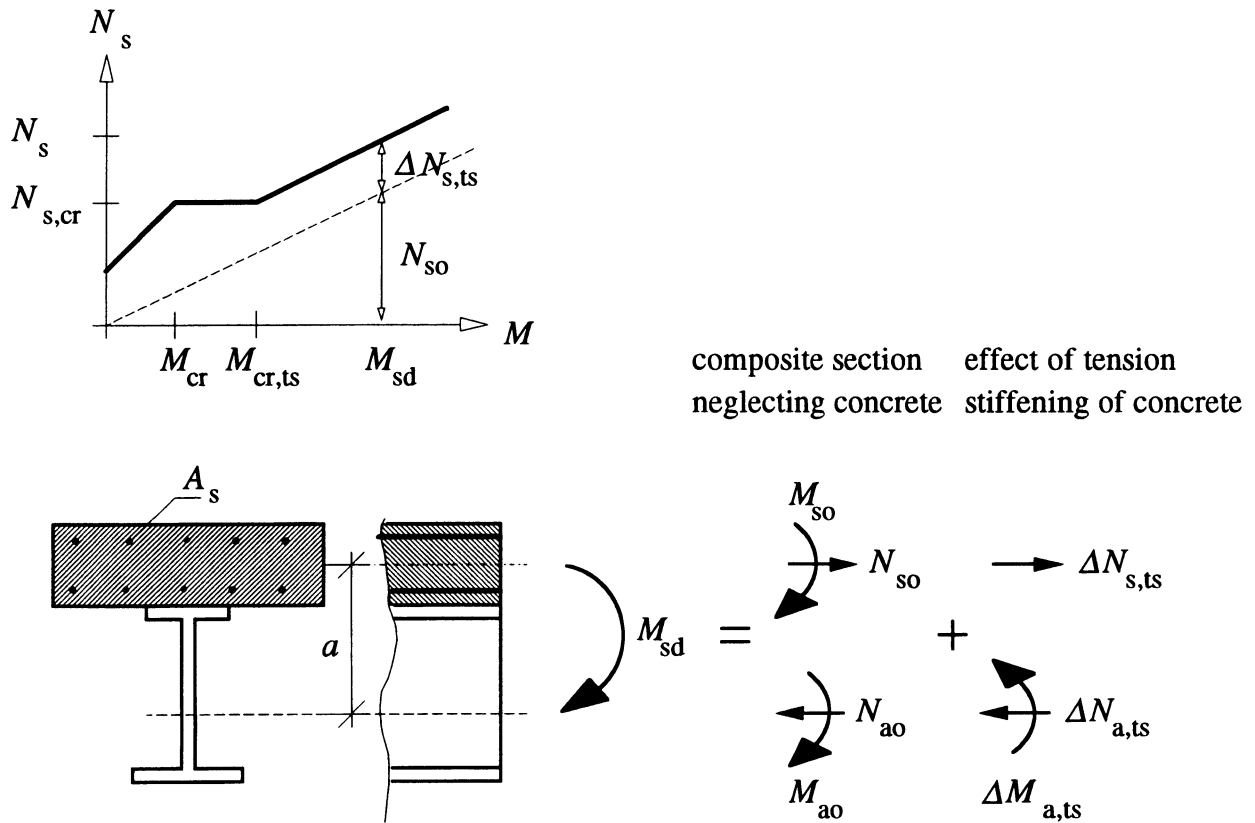


Figure L.5: Internal forces of the individual cross-sections for the stage of stabilized cracking, region (c)

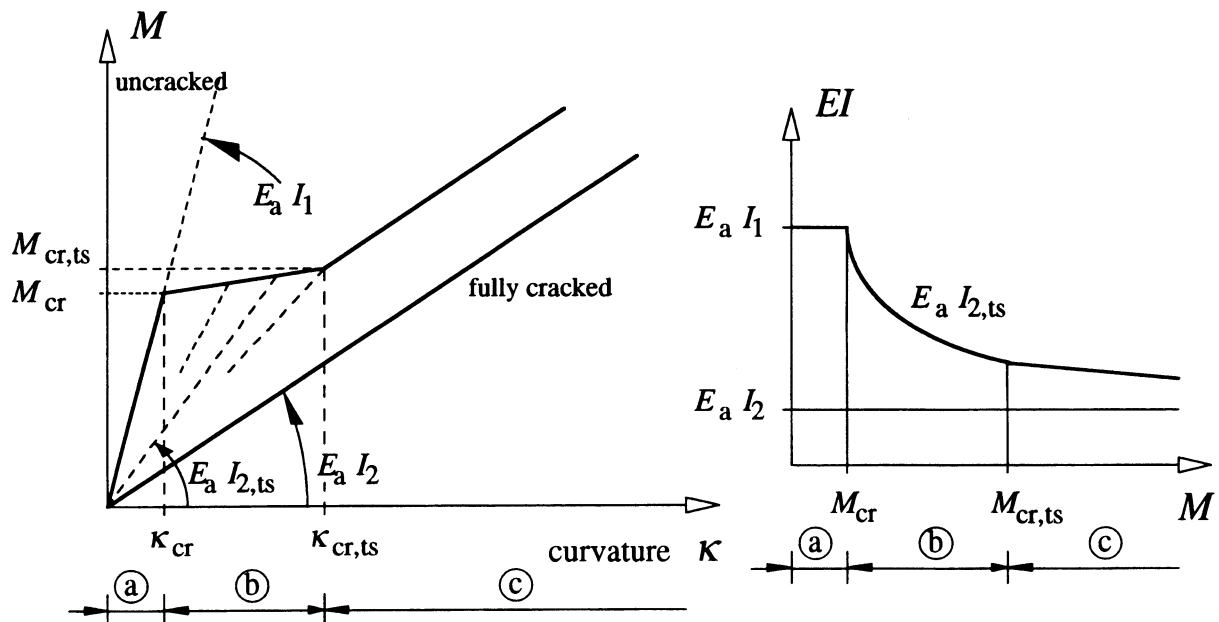


Figure L.6: Stiffness as a function of the bending moment M

L.4 Stiffness

(1) For the method given in 4.5.3.4 (5) (b) for the stiffness of a composite cross-section with concrete in tension and assumed to be cracked the contribution of concrete in tension between cracks should be taken into account. Figure L.6 demonstrates the stiffness in the different regions (a), (b) and (c) of a composite cross-section. The stiffnesses $E_a I_1$, $E_a I_2$ and $E_a I_{2,ts}$ are defined in 4.2.3 (2).

(2) The stiffness $E_a I_{2,ts}$ depends on the bending moment M acting on the composite section. For the method given in 4.5.3.4 (5) the bending moment M , calculated with the uncracked stiffness may be used. The stiffness $E_a I_{2,ts}$ may be calculated by:

$$E_a I_{2,ts} = \frac{E_a I_a}{1 - \frac{N_s \alpha}{M}}, \quad (\text{L.8})$$

where $E_a I_a$ is the stiffness of the structural steel section, M is the bending moment for the relevant load combination and N_s the tension force in the slab according to clause L.3, belonging to the moment M . In the stage of initial crack formation,

$$N_s = N_{s,cr} = A_{ct} f_{ctm} \frac{I}{I + \frac{h_c}{2z_0}} (1 + \rho_s n_0) \quad (\text{L.9})$$

may be used, where h_c and z_0 is given in 5.3.2.2 (2) and A_{ct} , f_{ctm} and ρ_s are defined in clause L.3.

L.5 Calculation of the stress range in reinforcing, prestressing and structural steel for fatigue loading

L.5.1 General

- (1) The stress ranges due to global effects should be calculated according to 4.12.4 (9).
- (2) For calculation of the maximum and minimum bending moments see 4.12.3 (2).

L.5.2 Stress ranges in reinforcing and prestressing steel

- (1) The maximum stress $\sigma_{\max,f,E}$ is given by (see figure L.7):

$$\sigma_{\max,f,E} = \sigma_{s,\max,EC} \frac{M_{\max,f,E}}{M_{\max,EC}} \alpha \quad (\text{L.10})$$

where:

$\sigma_{s,\max,EC}$ is the stress in the reinforcement or tendons calculated according to clause L.3 for the infrequent load combination,

$M_{\text{max,EC}}$ is the bending moment on the composite section for the infrequent load combination,

$M_{\max, f, E}$ is the maximum bending moment according to 4.12.3 (2),

α is a factor to take into account the different bond behaviour of reinforcing and prestressing steel with $\alpha = 1$ for prestressing steel and

$$\dot{a} = \frac{A_s + A_p}{A_s + \hat{I}_1 A_p} \quad (\text{L.11})$$

for reinforcement with A_s , A_p , and ξ_1 according to 5.3.3.2.

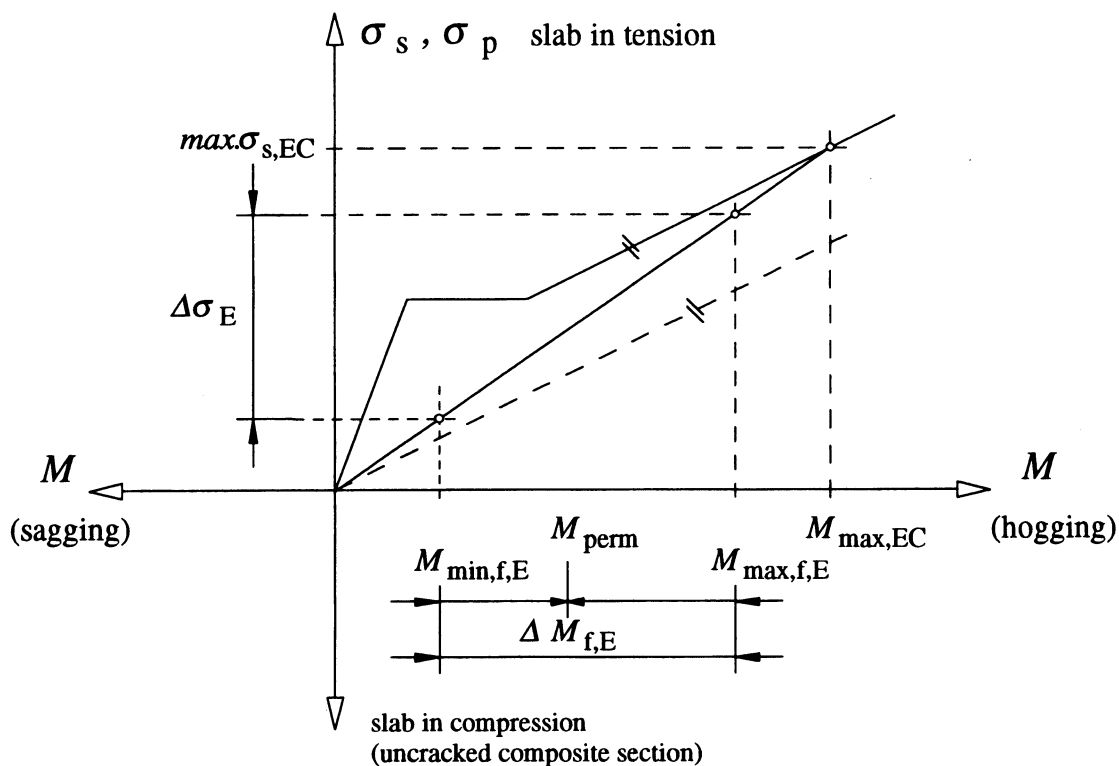


Figure L.7: Calculation of the stress range if $M_{\min,f,E}$ causes tension in the concrete slab

(2) The stress $\sigma_{\min.f.E}$ due to the bending moment $M_{\min.f.E}$ is given by:

$$\sigma_{\min, f, E} = \sigma_{s, \max, EC} \frac{M_{\min, f, E}}{M_{\max, EC}} \alpha \quad (\text{L.12})$$

where $M_{\min,f,E}$ is defined in 4.12.3(2) and the other symbols in (1). If $M_{\min,f,E}$ causes compressive stresses in the concrete slab the stress, $\sigma_{\min,f,E}$ should be determined with the cross-section properties of the uncracked section using the modular ratio for short term loading. For simplification $\sigma_{\min,f,E} = 0$ may be assumed.

(3) The stress ranges $\Delta\sigma_{s,E,loc}$ in reinforcing and prestressing steel caused by local effects should be calculated according to ENV 1992-2:1996.

L.5.3 Stress ranges in structural steel

- (1) The maximum stress $\sigma_{\max,f,E}$ is given by:

$$\sigma_{\max,f,E} = \sigma_{a,\max,EC} \frac{M_{\max,f,E}}{M_{\max,EC}} \quad (L.13)$$

where:

$\sigma_{a,\max,EC}$ is the stress in the structural steel section calculated according to clause L.3 for the infrequent load combination,

$M_{\max,EC}$ is the bending moment on the composite section for the infrequent load combination,

$M_{\max,f,E}$ is the maximum bending moment according to 4.12.3 (2).

- (2) If $M_{\min,f,E}$ acts in the same direction as $M_{\max,f,E}$, the stress $\sigma_{\min,f,E}$ is given by:

$$\sigma_{\min,f,E} = \sigma_{a,\max,EC} \frac{M_{\min,f,E}}{M_{\max,EC}} \quad (L.14)$$

where $M_{\min,f,E}$ is defined in 4.12.3(2) and the other symbols in (1). If $M_{\min,f,E}$ acts in the opposite direction to $M_{\max,f,E}$, and causes compression in the concrete slab the stress $\sigma_{\min,f,E}$ should be determined with the cross-section properties of the uncracked section using the modular ratio for short term loading.

L.5.4 Range of longitudinal shear per unit length, $\Delta v_{f,E}$ for shear connectors

- (1) The range of longitudinal shear $\Delta v_{f,E}$ between the steel section and the slab in regions where $M_{\max,f,E}$ and $M_{\min,f,E}$ causes tension in the concrete slab may be calculated from equation (L.15) taking into account the tension stiffening according to figures L.8. If $M_{\min,f,E}$ causes compression in the concrete slab the effect of tension stiffening on the longitudinal shear should be determined with figure L.9.

$$\Delta v_{f,E} = \frac{\Delta N_{s,f,E}}{\Delta M_{f,E}} (V_{\max,f,E} - V_{\min,f,E}) \quad (L.15)$$

where $V_{\max,f,E}$ and $V_{\min,f,E}$ are the vertical shear forces in the composite section due to the moments $M_{\max,f,E}$ and $M_{\min,f,E}$.

- (2) $\Delta M_{f,E} = M_{\max,f,E} - M_{\min,f,E}$ and $\Delta N_{s,f,E} = N_{s,\max,f,E} - N_{s,\min,f,E}$ should be calculated according to figure L.8 if $M_{\min,f,E}$ causes tension in the slab and according to figure L.9 if $M_{\min,f,E}$ causes compression in the slab.

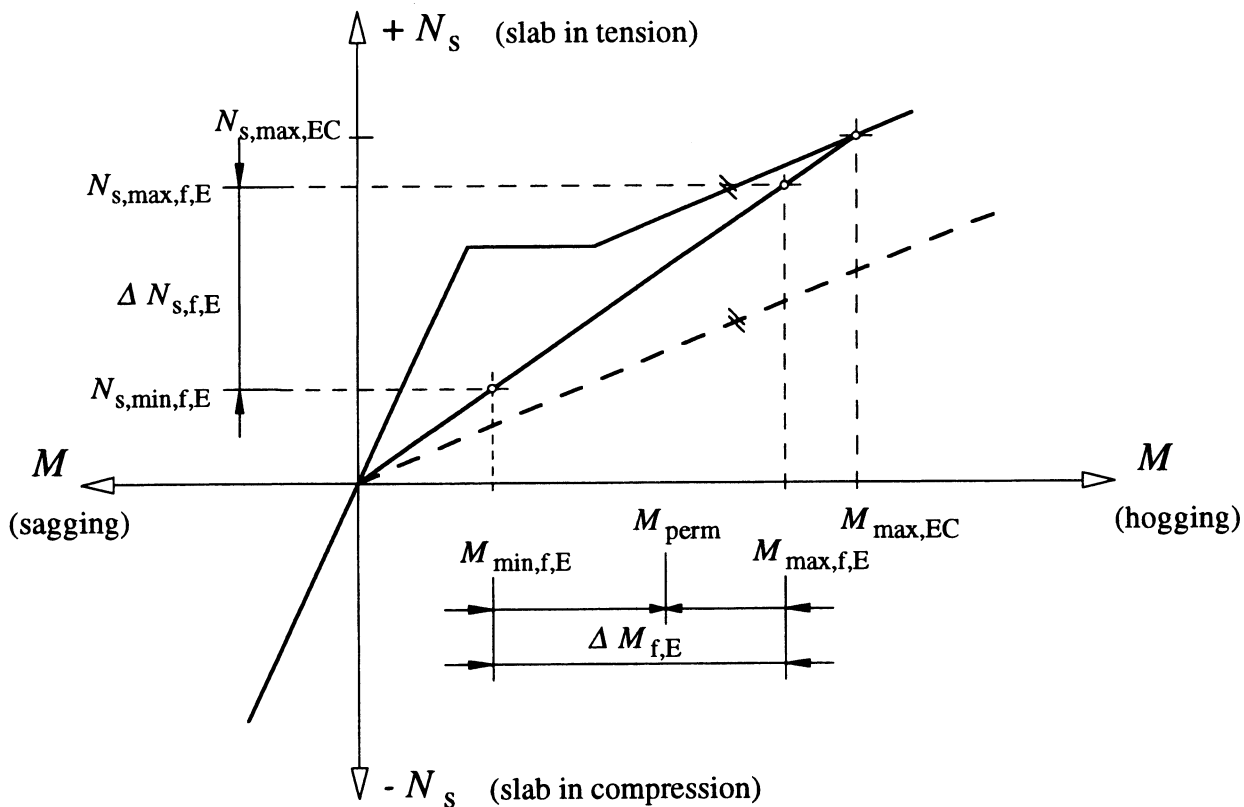


Figure L.8: Calculation of the stress range if $M_{min,f,E}$ is a bending moment with the slab in tension

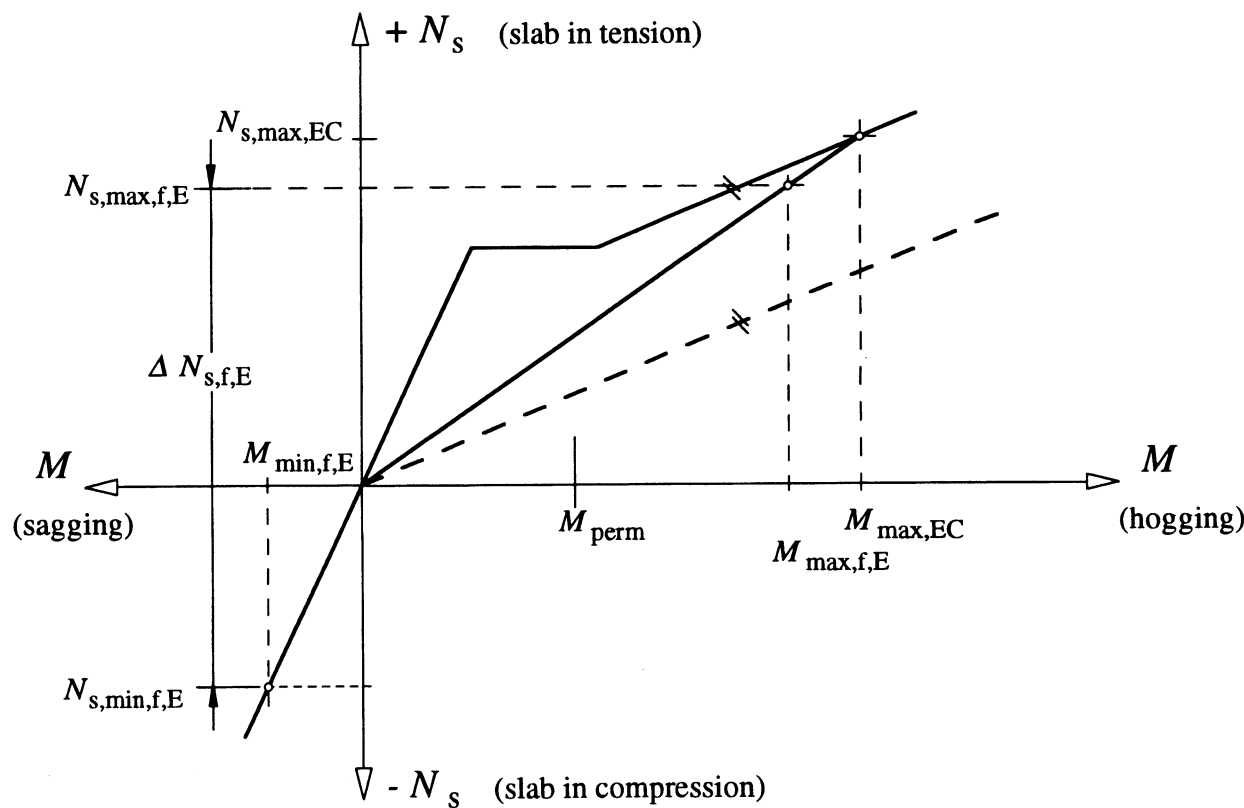


Figure L.9: Calculation of the stress range if $M_{min,f,E}$ is a bending moment with the slab in compression

BSI — British Standards Institution

BSI is the independent national body responsible for preparing British Standards. It presents the UK view on standards in Europe and at the international level. It is incorporated by Royal Charter.

Revisions

British Standards are updated by amendment or revision. Users of British Standards should make sure that they possess the latest amendments or editions.

It is the constant aim of BSI to improve the quality of our products and services. We would be grateful if anyone finding an inaccuracy or ambiguity while using this British Standard would inform the Secretary of the technical committee responsible, the identity of which can be found on the inside front cover.
Tel: +44 (0)20 8996 9000. Fax: +44 (0)20 8996 7400.

BSI offers members an individual updating service called PLUS which ensures that subscribers automatically receive the latest editions of standards.

Buying standards

Orders for all BSI, international and foreign standards publications should be addressed to Customer Services. Tel: +44 (0)20 8996 9001.
Fax: +44 (0)20 8996 7001. Email: orders@bsi-global.com. Standards are also available from the BSI website at <http://www.bsi-global.com>.

In response to orders for international standards, it is BSI policy to supply the BSI implementation of those that have been published as British Standards, unless otherwise requested.

Information on standards

BSI provides a wide range of information on national, European and international standards through its Library and its Technical Help to Exporters Service. Various BSI electronic information services are also available which give details on all its products and services. Contact the Information Centre.
Tel: +44 (0)20 8996 7111. Fax: +44 (0)20 8996 7048. Email: info@bsi-global.com.

Subscribing members of BSI are kept up to date with standards developments and receive substantial discounts on the purchase price of standards. For details of these and other benefits contact Membership Administration.
Tel: +44 (0)20 8996 7002. Fax: +44 (0)20 8996 7001.
Email: membership@bsi-global.com.

Information regarding online access to British Standards via British Standards Online can be found at <http://www.bsi-global.com/bsonline>.

Further information about BSI is available on the BSI website at <http://www.bsi-global.com>.

Copyright

Copyright subsists in all BSI publications. BSI also holds the copyright, in the UK, of the publications of the international standardization bodies. Except as permitted under the Copyright, Designs and Patents Act 1988 no extract may be reproduced, stored in a retrieval system or transmitted in any form or by any means – electronic, photocopying, recording or otherwise – without prior written permission from BSI.

This does not preclude the free use, in the course of implementing the standard, of necessary details such as symbols, and size, type or grade designations. If these details are to be used for any other purpose than implementation then the prior written permission of BSI must be obtained.

Details and advice can be obtained from the Copyright & Licensing Manager.
Tel: +44 (0)20 8996 7070. Fax: +44 (0)20 8996 7553.
Email: copyright@bsi-global.com.