EN 1993-2 (2006) (English): Eurocode 3: Design of steel structures - Part 2: Steel bridges [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

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EUROPEAN STANDARD

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English Version

Eurocode 3 - Design of steel structures - Part 2: Steel Bridges

Eurocode 3 - Calcul des structures en acier - Partie 2: Ponts métalliques

Eurocode 3 - Bemessung und konstruktion von Stahlbauten - Teil 2: Stahlbrücken

This European Standard was approved by CEN on 9 January 2006.

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EUROPEAN COMMITTEE FOR STANDARDIZATION
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EUROPÄISCHES KOMITEE FÜR NORMUNG

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Contents

Foreword

This European Standard EN 1993-2, Eurocode 3: Design of steel structures Part 2: Steel bridges, has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by April 2007 and conflicting National Standards shall be withdrawn at latest by March 2010.

This Eurocode supersedes ENV 1993-2.

According to the CEN-CENELEC Internal Regulations, the National Standard Organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to the CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products – CPD – and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode 0:	Basis of structural design
EN 1991	Eurocode 1:	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

[1] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1998 Eurocode 8: Design of structures for earthquake resistance

EN 1999 Eurocode 9: Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes :

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 - Mechanical resistance and stability - and Essential Requirement N°2 - Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from a harmonised product standard[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving a full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex (informative).

The National Annex (informative) may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e. :

- values for partial factors and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- geographical and climatic data specific to the Member State, e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

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[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for hENs and ETAGs/ETAs.

[3] According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

Links between Eurocodes and product harmonised technical specifications (ENs and ETAs)

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1993-2

EN 1993-2 is the second part of six parts of EN 1993 – Design of Steel Structures – and describes the principles and application rules for the safety and serviceability and durability of steel structures for bridges.

EN 1993-2 gives design rules which are supplementary to the generic rules in EN 1993-1-1.

EN 1993-2 is intended to be used with Eurocodes EN 1990 – Basis of design, EN 1991 – Actions on structures and the parts 2 of EN 1992 to EN 1998 when steel structures or steel components for bridges are referred to.

Matters that are already covered in those documents are not repeated.

EN 1993-2 is intended for use by

- committees drafting design related product, testing and execution standards,
- clients (e.g. for the formulation of their specific requirements),
- designers and constructors,
- relevant authorities.

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and quality management applies.

National annex for EN 1993-2

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-2 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-2 through:

- 2.1.3.2(1)
- 2.1.3.3(5)
- 2.1.3.4(1)
- 2.1.3.4(2)
- 2.3.1(1)
- 3.2.3(2)
- 3.2.3(3)
- 3.2.4(1)
- 3.4(1)
- 3.5(1)
- 3.6(1)

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[4] See Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

- 3.6(2)
- 4(1)
- 4(4)
- 5.2.1(4)
- 5.4.1(1)
- 6.1(1)P
- 6.2.2.3(1)
- AC1 6.2.2.5(1) AC1
- 6.3.2.3(1)
- 6.3.4.2(1)
- 6.3.4.2(7)
- 7.1(3)
- 7.3(1)
- 7.4(1)
- 8.1.3.2.1(1)
- 8.1.6.3(1)
- 8.2.1.4(1)
- 8.2.1.5(1)
- 8.2.1.6(1)
- 8.2.10(1)
- 8.2.13(1)
- 8.2.14(1)
- 9.1.2(1)
- 9.1.3(1)
- 9.3(1)P
- 9.3(2)P
- 9.4.1(6)
- 9.5.2(2)
- 9.5.2(3)
- 9.5.2(5)
- 9.5.2(6)
- 9.5.2(7)
- 9.5.3(2) (two places)
- 9.6(1) (two places)
- 9.7(1)
- A.3.3(1)P
- A.3.6(2)
- A.4.2.1(2)
- A.4.2.1(3)
- A.4.2.1(4)
- A.4.2.4(2)
- C.1.1(2)

- C.1.2.2(1)
- C.1.2.2(2)
- E.2(1)

1 General

1.1 Scope

1.1.1 Scope of Eurocode 3

(1) See 1.1.1(1), (2), (3), (4), (5) and (6) of EN 1993-1-1.

1.1.2 Scope of Part 2 of Eurocode 3

(1) EN 1993-2 provides a general basis for the structural design of steel bridges and steel parts of composite bridges. It gives provisions that supplement, modify or supersede the equivalent provisions given in the various parts of EN 1993-1.

(2) The design criteria for composite bridges are covered in EN 1994-2.

(3) The design of high strength cables and related parts are included in EN 1993-1-11.

(4) This European Standard is concerned only with the resistance, serviceability and durability of bridge structures. Other aspects of design are not considered.

(5) For the execution of steel bridge structures, EN 1090 should be taken into account.

NOTE: As long as EN 1090 is not yet available a provisional guidance is given in Annex C.

(6) Execution is covered to the extent that is necessary to indicate the quality of the construction materials and products that should be used and the standard of workmanship needed to comply with the assumptions of the design rules.

(7) Special requirements of seismic design are not covered. Reference should be made to the requirements given in EN 1998, which complements and modifies the rules of EN 1993-2 specifically for this purpose.

1.2 Normative references

(1) This European Standard incorporates, by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication applies (including amendments).

(2) In addition to the normative references given in EN 1990 and EN 1993-1 the following references should apply:

EN 1090	Execution of steel structures and aluminium structures
EN 1337	Structural bearings
EN 10029:1991	Specification for tolerances on dimensions, shape and mass for hot rolled steel plates 3 mm thick or above.
EN 10164	Steel products with improved deformation properties perpendicular to the surface of the product - Technical delivery conditions.
EN ISO 5817	Arc-welded joints in steel - Guidance on quality levels for imperfections.
EN ISO 12944-3	Paints and varnishes - Corrosion protection of steel structures by protective paint systems - Design considerations.
EN ISO 9013:2002	Thermal cutting - Classification of thermal cuts - Geometrical product specification and quality tolerances.

EN ISO 15613	Specification and qualification of welding procedures for metallic materials - Qualification based on pre-production welding test
EN ISO 15614-1	Specification and qualification of welding procedures for metallic materials - Welding procedure test - Part 1: Arc and gas welding of steels and arc welding of nickel and nickel alloys

1.3 Assumptions

(1) See 1.3(1) of EN 1993-1-1.

1.4 Distinction between principles and application rules

(1) See 1.4(1) of EN 1993-1-1.

1.5 Terms and definitions

(1) The terms and definitions given in EN 1990, EN 1993-1 and the following apply:

1.5.1
bridges
civil engineering construction works mainly intended to carry traffic or pedestrian loads over a natural obstacle or a communication line

NOTE: Railway bridges and bridges which carry canals, service pipes or other vehicles such as an aircraft are also covered.

1.5.2
abutment
any end support of a bridge

NOTE: A distinction is made between rigid abutments and flexible abutments where relevant.

1.5.3
integral abutment
abutment that is connected to the deck without any movement joint

1.5.4
pier
intermediate support of a bridge, situated under the deck

1.5.5
bearing
structural support located between the superstructure and an abutment or pier of the bridge that transfers loads from the deck to the abutment or pier

1.5.6
cable stay
tensioned element which connects the deck of a bridge to the pylon or pylons above the deck

1.5.7
prestress
permanent effect due to controlled forces and /or controlled deformations imposed within a structure

NOTE: Various types of prestress are distinguished from each other as relevant (such as prestress by tendons or prestress by imposed deformation of supports).

1.5.8
headroom
clear height available for traffic

1.5.9
breathing (of plates)
out-of-plane deformation of a plate caused by repeated application of in-plane loading

1.5.10
secondary structural elements
structural elements that do not form part of the main structure of the bridge

NOTE: The secondary structural elements are provided for other reasons, such as guard rails, parapets, ladders and access covers.

1.6 Symbols

(1) The symbols in EN 1990 und EN 1993-1 apply. Further symbols are given as follows:

$\sigma_{Ed,ser}$, $\tau_{Ed,ser}$	nominal stresses from the characteristic load combination
λ, λ_1, λ_2, λ_3, λ_4, λ_{max}, λ_{loc}, λ_{glo}	damage equivalent factors
Φ_2, Φ_{loc}, Φ_{glo}	damage equivalent impact factors
$\Delta\sigma_p$, $\Delta\sigma_{loc}$, $\Delta\sigma_{glo}$	stress ranges from load p
μ_k	characteristic value of friction coefficient
γ_μ	partial factor for friction
α	factor depending on type of bearing and number of bearings with adverse or relieving forces
T_{0max}, T_{0min}, T_0	temperatures
ΔT_0, ΔT_K, ΔT_γ	temperature differences
γ_T	partial factor for temperature
K, $K_{foundation}$, K_{pier}, $K_{bearing}$	spring stiffness
S_d, S_T	slide path

(2) Additional symbols are defined in the text where they first occur.

1.7 Conventions for member axes

(1) See 1.7(1), (2), (3) and (4) of EN 1993-1-1.

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1) See 2.1.1(1), (2) and (3) of EN 1993-1-1.

2.1.2 Reliability management

(1) See 2.1.2(1) of EN 1993-1-1.

2.1.3 Design working life, durability and robustness

2.1.3.1 General

(1) See 2.1.3.1(1) of EN 1993-1-1.

(2)P Bridges shall be designed for fatigue for the duration of their design working life.

2.1.3.2 Design working life

(1) The design working life should be taken as the period for which a bridge is required to be used for its intended purpose, taking into account anticipated maintenance but not major repair.

NOTE 1: The National Annex may specify the design working life. A design working life of a permanent bridge of 100 years is recommended.

NOTE 2: For temporary bridges the design working life may be stated in the project specifications.

(2) For structural elements that cannot be designed for the total design life of the bridge, see 2.1.3.3.

2.1.3.3 Durability

(1) To ensure durability, bridges and their components may be designed to minimise damage or be protected from excessive deformation, deterioration, fatigue and accidental actions that are expected during the design working life.

(2) Structural parts of a bridge to which guardrails or parapets are connected, should be designed to ensure that plastic deformations of the guardrails or parapets can occur without damaging the structure.

(3) Where a bridge includes components that need to be replaceable, see 4(6), the possibility of their safe replacement should be verified as a transient design situation.

(4) Permanent connections of structural parts of the bridge should be made with preloaded bolts in a Category B or C connection. Alternatively closely fitted bolts, rivets or welding may be used to prevent slipping.

(5) Joints where the transmission of forces is purely by contact may be used where justified by fatigue assessments.

NOTE: The National Annex may give additional recommendations for durable details.

2.1.3.4 Robustness and structural integrity

(1) The design of the bridge should ensure that when the damage of a component due to accidental actions occurs, the remaining structure can sustain at least the accidental load combination with reasonable means.

NOTE: The National Annex may define components that are subject to accidental design situations and also details for assessments. Examples of such components are hangers, cables, bearings.

(2) The effects of corrosion or fatigue of components and material should be taken into account by appropriate detailing, see also EN 1993-1-9 and EN 1993-1-10.

NOTE 1: EN 1993-1-9, section 3 provides assessment methods using the principles of damage tolerance or safe life.

NOTE 2: The National Annex may give a choice of the design method to be used for fatigue assessment.

NOTE 3: For guidance on access, maintenance and inspection, see section 4.

2.2 Principles of limit state design

(1) See 2.2(1) and (2) of EN 1993-1-1.

(3) For damage limitation at the ultimate limit state global analysis models should be elastic for transient and persistent design situations, see 5.4.

(4) The required fatigue life should be achieved through design for fatigue and/or appropriate detailing, see Annex C, and by serviceability checks.

2.3 Basic variables

2.3.1 Actions and environmental influences

(1) Actions for the design of bridges should be taken from EN 1991. For the combination of actions and partial factors for actions see Annex A.2 of EN 1990.

NOTE 1: For actions on steel bridge decks of road bridges, see Annex E.

NOTE 2: For actions not specified in EN 1991, see the National Annex.

(2) See 2.3(2), (3), (4) and (5) of EN 1993-1-1.

NOTE: For actions on bearings, see Annex A.

2.3.2 Material and product properties

(1) See 2.3.2(1) of EN 1993-1-1.

2.4 Verification by the partial factor method

(1) See 2.4.1(1), 2.4.2(1) and (2), 2.4.3(1) and 2.4.4(1) of EN 1993-1-1.

2.5 Design assisted by testing

(1) See 2.5(1), (2) and (3) of EN 1993-1-1.

3 Materials

3.1 General

(1) See 3.1(1) and (2) of EN 1993-1-1.

3.2 Structural steel

3.2.1 Material properties

(1) See 3.2.1(1) of EN 1993-1-1.

3.2.2 Ductility requirements

(1) See 3.2.2(1) and (2) of EN 1993-1-1.

3.2.3 Fracture toughness

(1) The material should have the required material toughness to prevent brittle fracture within the intended design working life of the structure.

(2) No further checks against brittle fracture need to be made if the conditions given in EN 1993-1-10 are met for the lowest service temperature.

NOTE 1: The lowest service temperature to be adopted in design may be taken from EN 1991-1-5.

NOTE 2: The National Annex may specify additional requirements depending on the plate thickness. An example is given in Table 3.1.

Table 3.1: Example for additional requirement for toughness of base material

Example	Nominal thickness	Additional requirement
1	$t \leq 30$ mm	T_{27J} = -20 °C in accordance with EN 10025
	$30 < t \leq 80$ mm	Fine grain steel in accordance with EN 10025, e.g. S355N/M
	$t > 80$ mm	Fine grain steel in accordance with EN 10025, e.g. S355NL/ML

(3) For bridge components under compression a suitable minimum toughness property should be selected.

NOTE: The National Annex may give guidance on the selection of toughness properties for members in compression. The use of Table 2.1 of EN 1993-1-10 for $\sigma_{Ed} = 0{,}25 f_y(t)$ is recommended.

3.2.4 Through thickness properties

(1) Steel with improved through thickness properties forming to EN 10164 should be used where required, see EN 1993-1-10.

NOTE: Where Z_{Ed} values have been determined in accordance with EN 1993-1-10, the required quality class according to EN 10164 may be chosen in the National Annex. The choice in Table 3.2 is recommended.

Table 3.2: Quality class conforming to EN 10164

Target value Z_{Ed}	Quality class
$Z_{Ed} \leq 10$	–
$10 < Z_{Ed} \leq 20$	Z15
$20 < Z_{Ed} \leq 30$	Z25
$Z_{Ed} > 30$	Z35

3.2.5 Tolerances

(1) The dimensional and mass tolerances of rolled steel sections, structural hollow sections and plates should conform with the relevant product standard, ETAG or ETA unless more severe tolerances are specified.

(2) For welded components the tolerances in EN 1090 should be applied.

(3) See 3.2.5(3) of EN 1993-1-1.

3.2.6 Design values of material coefficients

(1) See 3.2.6(1) of EN 1993-1-1.

3.3 Connecting devices

3.3.1 Fasteners

3.3.1.1 Bolts, nuts and washers

(1) Bolts, nuts and washers should conform to the Reference Standards given in EN 1993-1-8, [AC1⟩ 1.2.4: Group 4. ⟨AC1]

(2) The rules in this part are applicable to bolts of grades given in Table 3.3.

(3) The nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} are given in Table 3.3 and they should be adopted as characteristic values in calculations.

Table 3.3: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt grade	4.6	5.6	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	300	480	640	900
f_{ub} (N/mm^2)	400	500	600	800	1000

3.3.1.2 Preloaded bolts

(1) High strength structural bolts of bolt grades 8.8 and 10.9 which conform to the Reference standards given in EN 1993-1-8, [AC1⟩ 1.2.4: Group 4 ⟨AC1] may be used as preloaded bolts when controlled tightening is carried out in accordance with the Reference Standards given in EN 1993-1-8, [AC1⟩ 1.2.7: Group 7. ⟨AC1]

3.3.1.3 Rivets

(1) The material properties, dimensions and tolerances of steel rivets should conform to the Reference Standards given in EN 1993-1-8, [AC1⟩ 1.2.6: Group 6. ⟨AC1]

3.3.1.4 Anchor bolts

(1) The following steel grades may be used for anchor bolts:

- Steel grades in accordance with the appropriate Reference Standards given in EN 1993-1-8, [AC1⟩ 1.2.1: Group 1; ⟨AC1]
- Steel grades in accordance with the appropriate Reference Standards given in EN 1993-1-8, [AC1⟩ 1.2.4: Group 4; ⟨AC1]
- Reinforcing bars conforming to EN 10080.

The nominal yield strength for anchor bolts should not exceed 640 N/mm^2.

3.3.2 Welding consumables

(1) All welding consumables should conform to the Reference Standards given in EN 1993-1-8, [AC1⟩ 1.2.5: Group 5. ⟨AC1]

(2) The performance of the weld metal should not be less than the corresponding values specified for steel grade being welded. This should take into account:

- specified yield strength;
- ultimate tensile strength;
- elongation at failure;
- minimum Charpy V-notch energy value of the filler metal.

3.4 Cables and other tension elements

(1) For cables and other tension elements see EN 1993-1-11.

NOTE: The National Annex may specify the types of cables appropriate to the specific bridge types.

3.5 Bearings

(1) Bearings should conform to EN 1337.

NOTE: The National Annex may give guidance on the types of bearings applicable to bridges.

3.6 Other bridge components

(1) Expansion joints, guardrails, parapets and other ancillary items should conform to the relevant technical specifications.

NOTE: The National Annex may give guidance on the types of expansion joint, guardrail, parapet and other ancillary items applicable to bridges.

(2) The bridge deck surfacing system, the products used and the method of application should meet with the relevant technical specification.

NOTE: The National Annex may give guidance on the bridge deck surfacing system, the products used and the method of application applicable to bridges.

4 Durability

(1) See 4(1), (2) and (3) of EN 1993-1-1.

NOTE: The National Annex may give guidance on requirements for access to allow for inspection and maintenance.

(4) For elements that cannot be inspected fatigue checks should be carried out (see EN 1993-1-9) and appropriate corrosion allowances should be provided.

NOTE: The National Annex may give guidance on sealing against corrosion, measures to ensure air tightness of box girders or the provisions of extra steel thickness for inaccessible surfaces.

(5) The required fatigue life of the structure and its components should be attained by the:

- fatigue design of details in accordance with (1), (4) and EN 1993-1-9 and with serviceability checks carried out in accordance with section 7;
- structural detailing for orthotropic steel decks;
- material chosen in accordance with section 3;
- fabrication conforming to EN 1090.

(6) Components that cannot be designed with sufficient reliability to achieve the total design working life of the bridge should be replaceable. These may include:

- stays, cables, hangers;
- bearings;
- expansion joints;
- drainage devices;
- guardrails, parapets;
- asphalt layer and other surface protection;
- wind shields;
- noise barriers.

5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

(1) See 5.1.1(1), (2) and (3) of EN 1993-1-1.

(4) For the structural modelling and basic assumptions for components of bridges see EN 1993-1-1.

NOTE: For the design of plated components and cables see also EN 1993-1-5 and EN 1993-1-11.

5.1.2 Joint modelling

(1) See 5.1.2(1), (2), (3) and (4) of EN 1993-1-1 and EN 1993-1-8.

(5) For bridges, the type of joint and its modelling should be chosen to ensure that the required fatigue life can be attained.

NOTE: Rigid joints appropriate to the fatigue categories given in EN 1993-1-9 are suitable to be employed between members of bridges except for bearings or pinned connections or cables.

5.1.3 Ground structure interaction

(1) See 5.1.3(1) of EN 1993-1-1.

NOTE 2: The stiffness of the supports may be based on the deformation characteristics of the bearings, piers and foundation.

5.2 Global analysis

5.2.1 Effects of deformed geometry of the structure

(1) See 5.2.1(1), (2) and (3) of EN 1993-1-1.

AC1 (4) Bridges may be checked with first order theory if the following criterion is satisfied: AC1

$$\alpha_{cr} \geq 10 \tag{5.1}$$

where α_{cr} is defined in 5.2.1(3) of EN 1993-1-1

NOTE: The National Annex may give further guidance for the definition and calculation of α_{crit}.

(5) See 5.2.1(5) and (6) of EN 1993-1-1.

5.2.2 Structural stability of frames

(1) See 5.2.2(1), (2), (3) and (4) of EN 1993-1-1.

(5) Where the behaviour of a bridge or its components is governed by the first buckling mode (single degree of freedom) the second order effects M_{II} may be calculated by applying a factor to the bending moments M_I as follows:

$$M_{II} = M_I \frac{1}{1 - \frac{1}{\alpha_{cr}}} \tag{5.2}$$

where $\alpha_{cr} > 3$.

(6) See 5.2.2(7) and (8) of EN 1993-1-1.

5.3 Imperfections

5.3.1 Basis

(1) See 5.3.1(1), (2) and (3) of EN 1993-1-1.

5.3.2 Imperfections for global analysis of frames

(1) See 5.3.2(1), (2) and (3) of EN 1993-1-1.

NOTE 1: For piers α_m would be applicable, if cumulative effects from contributions of various piers occur (e.g. for piers forming a frame with the superstructure).

NOTE 2: For the use of member imperfections see also Annex D.

(4) See 5.3.2(6), (7), (8), (10) and (11) of EN 1993-1-1.

5.3.3 Imperfection for analysis of bracing systems

(1) See 5.3.3(1), (2), (3), (4) and (5) of EN 1993-1-1.

5.3.4 Member imperfections

(1) See 5.3.4(1), (2) and (3) of EN 1993-1-1.

5.4 Methods of analysis considering material non-linearities

5.4.1 General

(1) Elastic analysis should be used to determine the internal forces and moments for all persistent and transient design situations.

NOTE: The National Annex may give guidance to enable the user to determine when a plastic global analysis may be used for accidental design situations. For plastic global analysis see 5.4 and 5.5 of EN 1993-1-1.

5.4.2 Elastic global analysis

(1) See 5.4.2(1), (2) and (3) of EN 1993-1-1.

(4) If all sections are class 1 the effects of differential temperature, shrinkage and settlement at the ultimate limit state may be ignored.

5.5 Classification of cross sections

5.5.1 Basis

(1) See 5.5.1(1) of EN 1993-1-1.

5.5.2 Classification

(1) See 5.5.2(1), (2), (3), (4), (5), (6), (7), (8), (9) and (10) of EN 1993-1-1.

6 Ultimate limit states

6.1 General

(1)P The partial factors γ_M as defined in 2.4.3 of EN 1993-1-1 shall be applied to the various characteristic values of resistance in this section, see Table 6.1:

Table 6.1: Partial factors

a) resistance of members and cross section:	
– resistance of cross sections to excessive yielding including local buckling	γ_{M0}
– resistance of members to instability assessed by member checks	γ_{M1}
– resistance of cross sections in tension to fracture	γ_{M2}
b) resistance of joints	
– resistance of bolts – resistance of rivets – resistance of pins – resistance of welds – resistance of plates in bearing	γ_{M2}
– slip resistance – at ultimate limit state (Category C) – at serviceability limit state	 γ_{M3} $\gamma_{M3,ser}$
– bearing resistance of an injection bolt	γ_{M4}
– resistance of joints in hollow section lattice girders	γ_{M5}
– resistance of pins at serviceability limit state	$\gamma_{M6,ser}$
– preload of high strength bolts	γ_{M7}

NOTE 1: For the partial factor γ_c for the resistance of concrete see EN 1992.

NOTE 2: The partial factors γ_{Mi} for bridges may be defined in the National Annex. The following numerical values are recommended:

$\gamma_{M0} = 1,00$

$\gamma_{M1} = 1,10$

$\gamma_{M2} = 1,25$

$\gamma_{M3} = 1,25$

$\gamma_{M3,ser} = 1,10$

$\gamma_{M4} = 1,10$

$\gamma_{M5} = 1,10$

$\gamma_{M6,ser} = 1,00$

$\gamma_{M7} = 1,10$

6.2 Resistance of cross sections

6.2.1 General

(1) See 6.2.1(1), (2), (3), (4), (5), (6), (7), (8), (9) and (10) of EN 1993-1-1.

6.2.2 Section properties

6.2.2.1 Gross cross section

(1) See 6.2.1.1(1) of EN 1993-1-1.

6.2.2.2 Net area

(1) See 6.2.2.2(1), (2), (3), (4) and (5) of EN 1993-1-1.

6.2.2.3 Shear lag effects

(1) See 6.2.2.3(1) and (2) of EN 1993-1-1 and 3.2 and 3.3 of EN 1993-1-5.

NOTE: The National Annex may give guidance on the treatment of shear lag effects at the ultimate limit state.

6.2.2.4 Effective properties of cross section with class 3 webs and Class 1 or 2 flanges

(1) See 6.2.2.4(1) of EN 1993-1-1

6.2.2.5 Effects of local buckling for class 4 cross sections

(1) The effects of local buckling should be considered using one of the following two methods specified in EN 1993-1-5:

1. effective cross section properties of class 4 sections in accordance with EN 1993-1-5, section 4
2. limiting the stress levels to achieve cross section properties in accordance with EN 1993-1-5, section 10

NOTE: The National Annex may recommend which method is to be used. In case of the use of the method 2 the National Annex may give further guidance.

6.2.2.6 Effective cross section properties of class 4 sections

(1) See 6.2.2.5(1), (2), (3), (4) and (5) of EN 1993-1-1.

(2) For stress limits of circular hollow sections to conform to [AC1) class 4 section (AC1] properties, see EN 1993-1-6.

6.2.3 Tension

(1) See 6.2.3(1), (2), (3), (4) and (5) of EN 1993-1-1.

6.2.4 Compression

(1) See 6.2.4(1) of EN 1993-1-1.

(2) The design resistance of cross sections for uniform compression $N_{c,Rd}$ should be determined as follows:

a) without local buckling:

$$N_{c,Rd} = \frac{A\, f_y}{\gamma_{M0}} \quad \text{for class 1, 2 and 3 cross sections} \tag{6.1}$$

b) with local buckling:

$$N_{c,Rd} = \frac{A_{eff}\, f_y}{\gamma_{M0}} \quad \text{for class 4 cross sections or} \tag{6.2}$$

$$N_{c,Rd} = \frac{A\, \sigma_{limit}}{\gamma_{M0}} \quad \text{for stress limits} \tag{6.3}$$

where $\sigma_{limit} = \rho_x f_y$ is the limiting stress of the weakest part of the cross section in compression (see 10(5) of EN 1993-1-5)

(3) See 6.2.4(3) and (4) of EN 1993-1-1.

6.2.5 Bending moment

(1) See 6.2.5(1) of EN 1993-1-1.

(2) The design resistance for bending about the major axis should be determined as follows:

a) without local buckling:

$$M_{c,Rd} = \frac{W_{pl}\, f_y}{\gamma_{M0}} \quad \text{for class 1 and 2 cross sections} \tag{6.4}$$

$$M_{c,Rd} = \frac{W_{el,\min}\, f_y}{\gamma_{M0}} \quad \text{for class 3 cross sections} \tag{6.5}$$

b) with local buckling:

$$M_{c,Rd} = \frac{W_{eff,\min}\, f_y}{\gamma_{M0}} \quad \text{for class 4 cross sections or} \tag{6.6}$$

$$M_{c,Rd} = \frac{W_{el,\min}\, \sigma_{limit}}{\gamma_{M0}} \quad \text{for stress limits} \tag{6.7}$$

where $W_{el,min}$ and $W_{eff,min}$ are the elastic moduli which correspond to the fibre with the maximum elastic stress

σ_{limit} is the limiting stress of the weakest part of the cross section in compression (see 2.4 of EN 1993-1-5)

(3) See 6.2.5(3), (4), (5) and (6) of EN 1993-1-1.

6.2.6 Shear

(1) See 6.2.6(1), (2), (3), (4), (5), (6), (7) and (8) of EN 1993-1-1 and section 5 of EN 1993-1-5.

6.2.7 Torsion

6.2.7.1 General

(1) Torsional and distortional effects should be taken into account for members subject to torsion.

(2) The effects of transverse stiffness in the cross section, [AC1⟩, and/or of diaphragms ⟨AC1] that are built in to reduce distortional deformations, may be taken into account by considering an appropriate elastic model which is subject to the combined effect of bending, torsion and distortion.

(3) Distortional effects in the members may be disregarded where the effects from distortion, due to the transverse bending stiffness in the cross section and/or diaphragm action, do not exceed 10 % of the bending effects.

(4) Diaphragms should be designed to take into account the action effects resulting from their load distributing effect.

6.2.7.2 Torsion for which distortional effects may be neglected

(1) See 6.2.7(1), (2), (3), (4), (5), (6), (7), (8), and (9) of EN 1993-1-1.

6.2.8 Bending, axial load, shear and transverse loads

(1) The interaction between bending, axial load, shear and transverse loads may be determined using one of the following two methods:

1. Interaction methods given in 6.2.8 to 6.2.10.

   NOTE: For local buckling effects see EN 1993-1-5, section 4 to 7.

2. Interaction of stresses using the yielding criterion given in 6.2.1

   NOTE: For local buckling effects EN 1993-1-5, see section 10.

6.2.9 Bending and shear

(1) See 6.2.8(1), (2), (3), (4), (5) and (6) of EN 1993-1-1.

6.2.10 Bending and axial force

6.2.10.1 Class 1 and class 2 cross sections

(1) See 6.2.9.1(1), (2), (3), (4), (5) and (6) of EN 1993-1-1.

6.2.10.2 Class 3 cross sections

(1) See 6.2.9.2(1) of EN 1993-1-1.

(2) The following should be met for local buckling consideration, when the limit stress method is used:

$$\sigma_{x,Ed} \leq \frac{\sigma_{limit}}{\gamma_{M0}} \leq \frac{f_y}{\gamma_{M0}} \qquad (6.8)$$

where σ_{limit} should be determined from section 10 of EN 1993-1-5.

6.2.10.3 Class 4 cross sections

(1) See 6.2.9.3(1) and (2) of EN 1993-1-1.

6.2.11 Bending, shear and axial force

(1) See 6.2.10(1), (2) and (3) of EN 1993-1-1.

6.3 Buckling resistance of members

6.3.1 Uniform members in compression

6.3.1.1 Buckling resistance

(1) See 6.3.1.1(1), (2), (3) and (4) of EN 1993-1-1.

6.3.1.2 Buckling curves

(1) See 6.3.1.2(1), (2), (3) and (4) of EN 1993-1-1.

6.3.1.3 Slenderness for flexural buckling

(1) See 6.3.1.3(1) and (2) of EN 1993-1-1.

6.3.1.4 Slenderness for torsional and torsional flexural buckling

(1) See 6.3.1.4(1), (2) and (3) of EN 1993-1-1.

6.3.1.5 Use of class 3 section properties with stress limits

(1) As an alternative to using class 4 section properties given in equations (6.48), (6.49), (6.51) and (6.53) of EN 1993-1-1, class 3 section properties given in equations (6.47), (6.49), (6.50) and (6.52) of EN 1993-1-1, with stress limits in accordance with section 10 of EN 1993-1-5, may be used, see 6.2.2.5.

6.3.2 Uniform members in bending

6.3.2.1 Buckling resistance

(1) See 6.3.2.1(1), (2), (3) and (4) of EN 1993-1-1.

6.3.2.2 Lateral torsional buckling curves – General case

(1) See 6.3.2.2(1), (2) and (3) of EN 1993-1-1.

(4) Lateral torsional buckling effects may be ignored if the slenderness parameter $\bar{\lambda}_{LT} \leq 0{,}2$ or $\dfrac{M_{Ed}}{M_{crit}} \leq 0{,}04$.

6.3.2.3 Lateral torsional buckling curves for rolled sections or equivalent welded sections

(1) See 6.3.2.3(1) and (2) of EN 1993-1-1.

NOTE: The National Annex may give further information

6.3.3 Uniform members in bending and axial compression

(1) Unless second order analysis is carried out using the imperfections given in 5.3.2, the stability of uniform members subject to axial compression and bending in the plane of buckling should be checked in accordance with section 6.3.3 or 6.3.4 of EN 1993-1-1.

NOTE: As a simplification to equation (6.61) in 6.3.3 of EN 1993-1-1 the following condition may be used:

$$\frac{N_{Ed}}{\dfrac{\chi_y N_{Rk}}{\gamma_{M1}}} + \frac{C_{mi,o}\left(M_{y,Ed} + \Delta M_{y,Ed}\right)}{\dfrac{M_{y,Rk}}{\gamma_{M1}}} \leq 0{,}9 \tag{6.9}$$

where N_{Ed} is the design value of the compression force;

$M_{y,Ed}$ is the design value of the maximum moment about the y-y axis of the member obtained from first order analysis without considering imperfections;

$\Delta M_{y,Ed}$ is the moment due to the shift of the centroidal axis according to 6.2.10.3;

$C_{mi.o}$ is the equivalent moment factor, see Table A.2 of EN 1993-1-1;

χ_y is the reduction factors due to flexural buckling from 6.3.1.

6.3.4 General method for lateral and lateral torsional buckling of structural components

6.3.4.1 General method

(1) See 6.3.4(1), (2), (3) and (4) of EN 1993-1-1.

6.3.4.2 Simplified method

(1) See 6.3.2.4(1) of EN 1993-1-1.

NOTE: The National Annex may give the limit of application. The values $\bar{\lambda}_{c,0} = 0{,}2$ and $k_{fl} = 1{,}0$ (see 6.3.2.4(2) of EN-1993-1-1) are recommended.

(2) Truss chords and flanges in compression that are subject to lateral buckling may be verified by modelling the elements as a column subject to the compression force N_{Ed} and supported by continuous or discrete elastic restraint modelled as springs.

NOTE 1: Guidance for determining the stiffness of the restraint in the form of U-frames is given in Annex D.2.4.

NOTE 2: Where truss [AC1⟩ flanges and/or chords ⟨AC1] are restrained by U-frames, the U-frame members are subjected to forces induced by the restraint and the interaction of the U-frame and the flanges [AC1⟩ and/or chords ⟨AC1].

(3) The buckling mode and the elastic critical buckling load N_{cr} may be determined from an elastic critical buckling analysis. If continuous springs are used to represent the restraints which are basically discrete, the critical buckling load should not be taken as being larger than that corresponding to the buckling with nodes at the locations of the restraints.

(4) The safety verification may be carried out in accordance with 6.3.2 using

$$\bar{\lambda}_{LT} = \sqrt{\frac{A_{eff}\ f_y}{N_{crit}}} \qquad (6.10)$$

where A_{eff} is the effective area of the chord;

N_{crit} is the elastic critical load determined with A_{gross}.

(5) For chords in compression or the bottom flanges of continuous girders between rigid supports, the effect of initial imperfections and second order effects on a supporting spring may be taken into account by applying an additional lateral force F_{Ed} at the connection of the chord to the spring such that:

$$F_{Ed} = \frac{N_{Ed}}{100} \qquad if\ \ell_k \leq 1{,}2\ \ell$$

$$F_{Ed} = \frac{\ell}{\ell_k}\frac{N_{Ed}}{80}\frac{1}{1-\frac{N_{Ed}}{N_{crit}}} \qquad if\ \ell_k > 1{,}2\ \ell \qquad (6.11)$$

where $\ell_k = \pi\sqrt{\frac{EI}{N_{crit}}}$

ℓ is the distance between the springs.

(6) If the [AC1) design value of the compression force N_{Ed} is (AC1] constant over the length of the chord, the critical axial load N_{crit} may be calculated from

$$N_{crit} = m\, N_E \tag{6.12}$$

where $N_E = \pi^2 \dfrac{EI}{L^2}$

$$m = \frac{2}{\pi^2}\sqrt{\gamma} \text{ but not less than 1,0.}$$

$$\gamma = \frac{c\, L^4}{E\, I}$$

$$c = \frac{C_d}{\ell}$$

where L is the span length between the rigid supports;

ℓ is the distance between the springs

C_d is the spring stiffness, see (2), NOTE 1.

A lateral support to a compressed flange may be assumed to be rigid if its stiffness C_d satisfies:

$$C_d > \frac{4N_E}{L} \tag{6.13}$$

[AC1) where N_E is the elastic critical load (AC1] which is determined assuming hinged ends.

(7) The procedure given in (2) to (6) may also be applied to the flanges of girders in compression when A_{eff} in (4) is substituted by

$$A_{eff} + \frac{A_{wc}}{3}$$

where A_{wc} is the area of the compression zone of the web. In the case of a class 4 section the areas should be taken as the effective areas.

NOTE: The National Annex may give further guidance for the case where the compressive force N_{Ed} is not constant over the length of the chord. The following method is recommended.

For the bottom flange of a continuous girder with rigid lateral supports at a distance L (see Figure 6.1) m in equation (6.12) may be taken as the minimum value obtained from the two following values:

$$m = 1 + 0{,}44\,(1+\mu)\,\Phi^{1,5} + (3 + 2\,\Phi)\,\gamma/(350 - 50\mu) \tag{6.14}$$

$$m = 1 + 0{,}44\,(1+\mu)\,\Phi^{1,5} + (0{,}195 + (0{,}05 + \mu/100)\,\Phi)\,\gamma^{0,5}$$

where $\mu = V_2/V_1$, see Figure 6.1

$\Phi = 2\,(1 - M_2/M_1)/(1+\mu)$ for $M_2 > 0$

Where the bending moment changes signs, equation (6.14) may be used as a conservative estimate by inserting $M_2 = 0$.

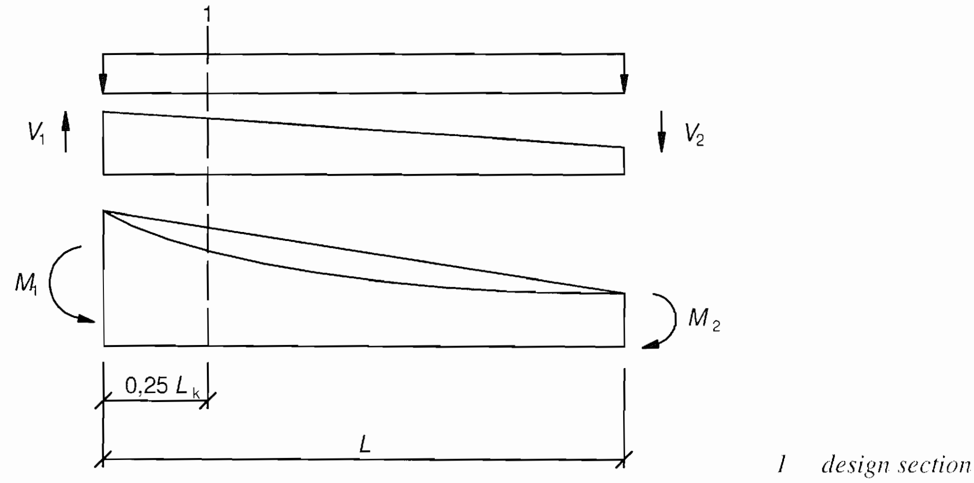


Figure 6.1: Segment of beam between rigid lateral supports with bending moment varying as a parabola

The verification of resistance to lateral torsional buckling in accordance with 6.3.2.2 may be carried out at a distance 0,25 L_k from the support with the largest moment as shown in Figure 6.1, provided that the cross-sectional resistance is also checked at the section with the largest moment, where $L_k = L/\sqrt{m}$.

6.4 Built-up compression members

(1) See section 6.4 of EN 1993-1-1.

6.5 Buckling of plates

(1) For buckling of plates in a fabricated girder the rules given in EN 1993-1-5 should be applied.

(2) The plate buckling verification of members at the ultimate limit state should be carried out using either a) or b) as follows:

a) Direct stresses, shear stresses and transverse forces should be verified according to section 4, 5 or 6 of EN 1993-1-5. Additionally, the interaction criteria in section 7 of EN 1993-1-5 should also be met.

b) Reduced stress method on the basis of stress limits governed by local buckling according to section 10 of EN 1993-1-5.

NOTE: See also 6.2.2.5.

(3) For web stiffeners or stiffened deck plates which are subjected to compression and additional bending moments from loads transverse to the plane of the stiffened plate, the stability may be verified in accordance with 6.3.2.3.

7 Serviceability limit states

7.1 General

(1) See 7.1(1), (2) and (3) of EN 1993-1-1.

(4) The following serviceability criteria should be met:

a) Restriction to elastic behaviour in order to limit:

- excessive yielding, see 7.3(1);
- deviations from the intended geometry by residual deflections, see 7.3(1);
- excessive deformations, see 7.3(4).

b) Limitation of deflections and curvature in order to prevent:

- unwanted dynamic impacts due to traffic (combination of deflection and natural frequency limitations), see 7.7 and 7.8;
- infringement of required clearances, see 7.5 or 7.6;
- cracking of surfacing layers, see 7.8;
- damage of drainage, see 7.12.

c) Limitation of natural frequencies, see 7.8 and 7.9, in order to:

- exclude vibrations due to traffic or wind which are unacceptable to pedestrians or passengers in cars using the bridge;
- limit fatigue damages caused by resonance;
- limit excessive noise emission.

d) Restriction of plate slenderness, see 7.4, in order to limit:

- excessive rippling of plates;
- breathing of plates;
- reduction of stiffness due to plate buckling, resulting in an increase of deflection, see EN 1993-1-5.

e) Improved durability by appropriate detailing to reduce corrosion and excessive wear, see 7.11.

f) Ease of maintenance and repair, see 7.11, to ensure:

- accessibility of structural parts for maintenance and inspection, renewal of corrosion protection and asphaltic pavements;
- replacement of bearings, anchors, cables, expansion joints with minimum disruption to the use of the structure.

(5) In most situations serviceability aspects should be dealt with in the conceptual design of the bridge, or by suitable detailing. However in appropriate cases, serviceability limit states may be verified by numerical assessment, e.g. for calculating deflections or eigen frequencies.

NOTE: The National Annex may give guidance on serviceability requirements for specific types of bridges.

7.2 Calculation models

(1) Stresses at serviceability limit states should be determined from a linear elastic analysis, using the appropriate section properties, see EN 1993-1-5.

(2) In modelling the structure, the non-uniform distribution of loads and stiffness resulting from the changes in plate thickness, stiffening etc. should be taken into account.

(3) Deflections should be determined by linear elastic analysis using the appropriate section properties, see EN 1993-1-5.

NOTE: Simplified calculation models may be used for stress calculations provided that the effects of the simplification are conservative.

7.3 Limitations for stress

(1) The nominal stresses $\sigma_{Ed,ser}$ and $\tau_{Ed,ser}$ resulting from the characteristic load combinations calculated making due allowance for the effects of shear lag in flanges and the secondary effects caused by deflections (e.g. secondary moments in trusses), should be limited as follows:

$$\sigma_{Ed,ser} \leq \frac{f_y}{\gamma_{M,ser}} \tag{7.1}$$

$$\tau_{Ed,ser} \leq \frac{f_y}{\sqrt{3}\,\gamma_{M,ser}} \tag{7.2}$$

$$\sqrt{\sigma_{Ed,ser}^2 + 3\tau_{Ed,ser}^2} \leq \frac{f_y}{\gamma_{M,ser}} \tag{7.3}$$

NOTE 1: Where relevant the above checks should include stresses σ_z from transverse loads, see EN 1993-1-5.

NOTE 2: The National Annex may give the value for $\gamma_{M,ser}$. $\gamma_{M,ser} = 1{,}00$ is recommended.

NOTE 3: Plate buckling effects may be ignored as specified in EN 1993-1-5, 2.2(5).

(2) The nominal stress range $\Delta\sigma_{fre}$, due to the frequent load combination should be limited to $1{,}5\, f_y/\gamma_{M,ser}$, see EN 1993-1-9.

(3) For non-preloaded bolted connections subject to shear, the bolt forces due to the characteristic load combination should be limited to:

$$F_{b,Rd,ser} \leq 0{,}7\, F_{b,Rd} \tag{7.4}$$

where $F_{b,Rd}$ is the bearing resistance for ultimate limit states verifications.

(4) For slip-resistant preloaded bolted connections category B (slip resistant at serviceability, see EN 1993-1-8), the assessment for serviceability should be carried out using the characteristic load combination.

7.4 Limitation of web breathing

(1) The slenderness of web plates should be limited to avoid excessive breathing that might result in fatigue at or adjacent to the web-to-flange connections.

NOTE: The National Annex may define cases where web breathing checks are not necessary.

(2) Web breathing may be neglected for web panels without longitudinal stiffeners or for subpanels of stiffened webs, where the following criteria are met:

$$b/t \leq 30 + 4{,}0\, L \quad \leq 300 \qquad \text{for road bridges} \tag{7.5}$$

$$b/t \leq 55 + 3{,}3\, L \quad \leq 250 \qquad \text{for railway bridges} \tag{7.6}$$

where L is the span length in m, but not less than 20 m.

(3) If the provision in (2) is not satisfied web breathing should be checked as follows:

$$\sqrt{\left(\frac{\sigma_{x,Ed,ser}}{k_\sigma\,\sigma_E}\right)^2+\left(\frac{1{,}1\,\tau_{x,Ed,ser}}{k_\tau\,\sigma_E}\right)^2}\leq 1{,}1 \qquad (7.7)$$

where $\sigma_{x,Ed,ser}$, $\tau_{Ed,ser}$ are the stresses for the frequent load combination. If the stresses are not uniform along the length of the panel, see section 4.6(3) of EN 1993-1-5;

k_σ, k_τ are the linear elastic buckling coefficients assuming hinged edges of the panel, see EN 1993-1-5;

AC1 $$\sigma_E = 190000\left(\frac{t}{b_p}\right)^2 \quad [N/mm^2]$$ AC1

b_p is the smaller of a and b.

NOTE: For stresses varying along the panel see EN 1993-1-5, 4.6(3).

7.5 Limits for clearance gauges

(1) Specified clearance gauges should be maintained without encroachment by any part of the structure under the effects of the characteristic load combination.

7.6 Limits for visual impression

(1) To achieve a satisfactory appearance of the bridge consideration should be given to precambering.

(2) In calculating camber the effects of shear deformation and slip in riveted or bolted connections should be considered.

(3) For connections with rivets or fitted bolts, a fastener slip of 0,2 mm should be assumed. For preloaded bolts, slip does not need to be considered.

7.7 Performance criteria for railway bridges

(1) Specific criteria for deflection and vibrations for railway bridges should be obtained from EN 1991-2.

(2) Any requirements for the limitation of noise emission may be given in the project specification.

7.8 Performance criteria for road bridges

7.8.1 General

(1) Excessive deformations should be avoided where it could:

- endanger traffic by excessive transverse slope when the surface is iced;
- affect the dynamic load on the bridge by impact from wheels;
- affect the dynamic behaviour causing discomfort to users;
- lead to cracks in asphaltic surfacings;
- adversely affect the drainage of water from the bridge deck.

NOTE: For durability requirements see Annex C.

(2) Deformations should be calculated using the frequent load combination.

(3) The natural frequency of vibrations and deflections of the bridge should be limited to avoid discomfort to users.

7.8.2 Deflection limits to avoid excessive impact from traffic

(1) The deck structure should be designed to ensure that its deflection along the length is uniform and that there is no abrupt change in cross section giving rise to impact. Sudden changes in the slope of the deck and changes of level at the expansion joints should be eliminated. Any transverse girders at the end of the bridge should be designed to ensure that the deflection does not exceed:

- the limit specified for the proper functioning of the expansion joint;
- 5 mm under frequent loads unless other limits are specified for the particular type of expansion joint.

NOTE: Guidance on the deflection limit of expansion joints is given in Annex B.

(2) Where the deck structure is irregularly supported (e.g. by additional bracings at intermediate bridge piers), the deck area adjacent to these additional deck supports should be designed for the enhanced impact factors given in EN 1991-2 for the area close to the expansion joints.

7.8.3 Resonance effects

(1) Mechanical resonance should be taken into account when relevant. Where light bracing members, cable stays or similar components have natural frequencies that are close to the frequency of any mechanical excitation due to regular passage of vehicles over deck joints, consideration should be given to either increasing the stiffness or providing artificial dampers, i.e. oscillation dampers.

NOTE: Guidance on members supporting expansion joints is given in Annex B.

7.9 Performance criteria for pedestrian bridges

(1) For footbridges and cycle bridges with excessive vibrations could cause discomfort to users, measures should be taken to minimise such vibrations by designing the bridge with appropriate natural frequency or by providing suitable damping devices.

7.10 Performance criteria for the effect of wind

(1) Vibrations of slender members induced by vortex excitation should be minimised to prevent repetitive stresses of sufficient magnitude that could cause fatigue.

NOTE: Guidance on the determination of fatigue loads from vortex excitation is given in EN 1991-1-4.

7.11 Accessibility of joint details and surfaces

(1) All steelwork should be designed and detailed to minimise the risk of corrosion and to permit inspection and maintenance, see ISO 12944-3.

(2) All parts should normally be designed to be accessible for inspection, cleaning and painting. Where such access is not possible, all inaccessible parts should either be effectively sealed against corrosion (e.g. the interior of boxes or hollow portions) or they should be constructed in steel with improved atmospheric corrosion resistance. Where the environment or access provisions are such that corrosion can occur during the life of the bridge, a suitable allowance for this should be made in the proportioning of the parts see 4(4).

7.12 Drainage

(1) All decks should be waterproofed and the surfaces of carriageways and footpaths should be sealed to prevent the ingress of water.

(2) The layout of the drainage should take into account the slope of the bridge deck as well as the location, diameter and slope of the pipes.

(3) Free fall drains should carry water to a point clear of the underside of the structure to prevent water entering into the structure.

(4) Drainage pipes should be designed so that they can be cleaned easily . The distance between centres of the cleaning openings should be shown on drawings.

(5) Where drainage pipes are used inside box girder bridges, provisions should be made to prevent accumulation of water during leaks or breakage of pipes.

(6) For road bridges, drains should be provided at expansion joints on both sides where is appropriate.

(7) For railway bridges up to 40 m long carrying ballasted tracks, the deck may be assumed to be self-draining to abutment drainage systems and no further drainage provisions need to be provided along the length of the deck.

(8) Provision should be made for the drainage of all closed cross sections, unless these are fully sealed by welding.

8 Fasteners, welds, connections and joints

8.1 Connections made of bolts, rivets and pins

8.1.1 Categories of bolted connections

8.1.1.1 Shear connections

(1) See 3.4.1(1) of EN 1993-1-8.

8.1.1.2 Tension connections

(1) See 3.4.2(1) of EN 1993-1-8.

8.1.2 Positioning of holes for bolts and rivets

(1) See 3.5(1) and (2) of EN 1993-1-8.

8.1.3 Design resistance of individual fasteners

8.1.3.1 Bolts and rivets

(1) See 3.6.1(1), (2), (3), (4), (5), (6), (7), (8), (9), (10), (11), (12), (13), (14), (15) and (16) of EN 1993-1-8.

8.1.3.2 Injection bolts

8.1.3.2.1 General

(1) See 3.6.2.1(1) and (2) of EN 1993-1-8.

NOTE: The National Annex may give guidance on the use of injection bolts.

8.1.3.2.2 Design resistance

(1) See 3.6.2.2(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.1.4 Groups of fasteners

(1) See 3.7(1) of EN 1993-1-8.

8.1.5 Long joints

(1) See 3.8(1) and (2) of EN 1993-1-8.

8.1.6 Slip resistant connections using 8.8 and 10.9 bolts

8.1.6.1 Slip resistance

(1) See 3.9.1(1) and (2) of EN 1993-1-8.

8.1.6.2 Combined tension and shear

(1) See 3.9.2(1) and (2) of EN 1993-1-8.

8.1.6.3 Hybrid connections

(1) See 3.9.3(1) of EN 1993-1-8.

NOTE: The National Annex may give guidance on the use of hybrid connections.

8.1.7 Deductions for fastener holes

8.1.7.1 General

(1) See 3.10.1(1) of EN 1993-1-8.

8.1.7.2 Design for block tearing

(1) See 3.10.2(1), (2) and (3) of EN 1993-1-8.

8.1.7.3 Angles connected by one leg and other unsymmetrically connected members in tension

(1) See 3.10.3(1) and (2) of EN 1993-1-8.

8.1.7.4 Lug angles

(1) See 3.10.4(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.1.8 Prying forces

(1) See 3.11(1) of EN 1993-1-8.

8.1.9 Distribution of forces between fasteners at the ultimate limit state

(1) If a moment is applied to a joint, the distribution of internal forces should be linearly proportional to the distance from the centre of rotation.

(2) See 3.12(3) of EN 1993-1-8.

8.1.10 Connections made with pins

8.1.10.1 General

(1) See 3.13.1(1), (2), (3) and (4) of EN 1993-1-8.

8.1.10.2 Design of pins

(1) See 3.13.2(1), (2) and (3) of EN 1993-1-8.

8.2 Welded connections

8.2.1 Geometry and dimensions

8.2.1.1 Type of weld

(1) See 4.3.1(1) and (2) of EN 1993-1-8.

8.2.1.2 Fillet welds

8.2.1.2.1 General

(1) See 4.3.2.1(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.2.1.2.2 Intermittent fillet welds

(1) Intermittent fillet weld should not be used at locations, where they could result in the possible formation of rust pockets.

NOTE: Where the connection is protected from weather, e.g. in the interior of box sections, intermittent fillet welds are permitted.

8.2.1.3 Fillet welds all round

(1) See 4.3.3(1), (2), (3) and (4) of EN 1993-1-8.

8.2.1.4 Butt welds

(1) See 4.3.4(1), (2) and (3) of EN 1993-1-8.

NOTE: The National Annex may give guidance on the use of partial penetration butt welds.

8.2.1.5 Plug welds

(1) See 4.3.5(1) of EN 1993-1-8.

NOTE: The National Annex may give further guidance on the use of plug welds.

(2) See 4.3.5(2), (3), (4) and (5) of EN 1993-1-8.

8.2.1.6 Flare groove welds

(1) See 4.3.6(1) of EN 1993-1-8.

NOTE: The National Annex may give further guidance on the use of flare groove welds.

8.2.2 Welds with packings

(1) See 4.4(1), (2) and (3) of EN 1993-1-8.

8.2.3 Design resistance of a fillet weld

(1) For the design resistance of a fillet weld see 4.5 of EN 1993-1-8.

8.2.4 Design resistance of fillet welds all round

(1) See 4.6(1) of EN 1993-1-8.

8.2.5 Design resistance of butt welds

8.2.5.1 Full penetration butt welds

(1) See 4.7.1(1) of EN 1993-1-8.

8.2.5.2 Partial penetration butt welds

(1) See [AC1⟩ 4.7.2(1) and (2) of EN 1993-1-8.⟨AC1]

8.2.5.3 T-butt joints

(1) See 4.7.3(1) and (2) of EN 1993-1-8.

8.2.6 Design resistance of plug welds

(1) See 4.8(1) and (2) of EN 1993-1-8.

8.2.7 Distribution of forces

(1) See 4.9(1), (2), (3), (4), (5) and (6) of EN 1993-1-8.

8.2.8 Connections to unstiffened flanges

(1) See [AC1⟩ 4.10(1), (2), (3), (4) and (5) of EN 1993-1-8. ⟨AC1]

8.2.9 Long joints

(1) See 4.11(1), (2), (3) and (4) of EN 1993-1-8.

8.2.10 Eccentrically loaded single fillet or single-sided partial penetration butt welds

(1) See 4.12(1) and (2) of EN 1993-1-8.

NOTE: The National Annex may give further guidance on the use of eccentrically loaded single fillet or single sided partial penetration butt welds.

8.2.11 Angles connected by one leg

(1) See 4.13(1), (2) and (3) of EN 1993-1-8.

8.2.12 Welding in cold-formed zones

(1) See 4.14(1) of EN 1993-1-8.

8.2.13 Analysis of structural joints connecting H- and I-sections

(1) For the analysis of structural joints connecting H- and I-sections at the ultimate limit state see sections 5 and 6 of EN 1993-1-8.

NOTE: The National Annex may give further guidance on the use of structural joints connecting H- and I-sections.

8.2.14 Hollow section joints

(1) For the analysis of structural joints connecting hollow sections at the ultimate limit state see section 7 of EN 1993-1-8.

NOTE: The National Annex may give further guidance on the use of structural joints connecting hollow sections.

9 Fatigue assessment

9.1 General

9.1.1 Requirements for fatigue assessment

(1) Fatigue assessments should be carried out for all critical areas in accordance with EN 1993-1-9.

(2) Fatigue assessment is not applicable to:

- pedestrian bridges, bridges carrying canals or other bridges that are predominantly statically loaded, unless such bridges or parts of them are likely to be excited by wind loads or pedestrians;
- parts of railway or road bridges that are neither stressed by traffic loads nor likely to be excited by wind loads.

9.1.2 Design of road bridges for fatigue

(1) Fatigue assessments should be carried out for all bridge components unless the structural detailing complies with standard requirements for durable structures established through testing.

NOTE: The National Annex may give guidance on the conditions where no fatigue assessment is necessary.

(2) Fatigue assessment should be carried out using the procedure given in this section and EN 1993-1-9.

9.1.3 Design of railway bridges for fatigue

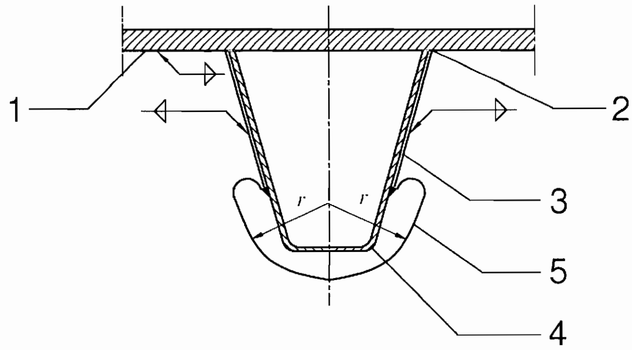
(1) Fatigue assessments should be carried out for all structural elements including the components listed in (2).

NOTE: Elements for which no assessment is needed may be given in the National Annex.

(2) For the bridge deck the following components should be checked:

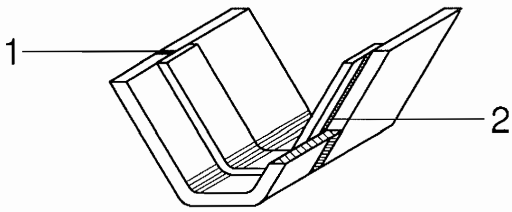
1. for bridge decks with longitudinal stiffeners and crossbeams
   - deckplate
   - stiffeners
   - crossbeams
   - stiffener to crossbeam connections
2. for bridge decks with transverse stiffeners only
   - deckplate
   - stiffeners

(3) For critical areas for fatigue checks see Figure 9.1 and Figure 9.2 and Table 9.8.



1 region 1
2 region 2
3 region 3
4 region 4 (splice)
5 region 5

Figure 9.1: Critical regions for fatigue, see also Table 9.8



1 butt weld
2 tack weld continuous along the full length of backing strip

Figure 9.2: Stiffeners with splice plates and metallic backing strips

9.2 Fatigue loading

9.2.1 General

(1) The fatigue loading from traffic should be obtained from EN 1991-2.

(2) The fatigue loads on slender elements due to wind excitations should be obtained from EN 1991-1-4.

9.2.2 Simplified fatigue load model for road bridges

(1) For the fatigue assessment of road bridges the fatigue load model 3 (single vehicle model) in conjunction with the traffic data specified for the bridge location in accordance with EN 1991-2 should be applied.

NOTE: See also 9.4.1(6).

9.2.3 Simplified fatigue load model for railway bridges

(1) For the fatigue assessment of railway bridges the characteristic values for load model 71 should be used, including the dynamic factor Φ_2 given in EN 1991-2.

9.3 Partial factors for fatigue verifications

(1)P The partial factor for fatigue loads shall be taken as γ_{Ff}.

NOTE: The National Annex may give the value for γ_{Ff}. The use of $\gamma_{Ff} = 1{,}0$ is recommended.

(2)P The partial factor for fatigue resistance shall be taken as γ_{Mf}.

NOTE: The National Annex may give values for γ_{Mf}. The values given in Table 3.1 of EN 1993-1-9 are recommended.

9.4 Fatigue stress range

9.4.1 General

(1) For the simplified fatigue loading specified in 9.2.2 or 9.2.3, the following procedure may be used to determine the design stress range.

(2) The maximum stress $\sigma_{P,max}$ and the minimum stress $\sigma_{P,min}$ should be determined by evaluating influence areas.

(3) The reference stress range $\Delta\sigma_p$ for determining the damage effects of the stress range spectrum should be obtained from:

$$\Delta\sigma_p = | \sigma_{p,max} - \sigma_{p,min} | \tag{9.1}$$

(4) The damage effects of the stress range spectrum may be represented by the damage equivalent stress range related to 2×10^6 cycles:

AC1 $$\Delta\sigma_{E,2} = \lambda\Phi_2\Delta\sigma_p \tag{9.2}$$ AC1

where λ is the damage equivalence factor as defined in 9.5;

Φ_2 is the damage equivalent impact factor.

(5) For railway bridges the value of Φ_2 should be obtained from EN 1991-2. For road bridges Φ_2 may be taken as equal to 1,0, as it is included in the fatigue load model.

(6) As an alternative to the procedure given above, AC1 stress-range spectra AC1 may be obtained from the evaluation of stress history from the fatigue load vehicles as specified in EN 1991-2, see EN 1993-1-9.

NOTE: The National Annex may give guidance on the use of EN 1991-2.

9.4.2 Analysis for fatigue

9.4.2.1 Longitudinal stiffeners

(1) Longitudinal stiffeners should be analysed using a model for the integral structure or for simplicity, as continuous beams on elastic supports.

NOTE: For railway bridges longitudinal stiffeners may be analysed as continuous beams on elastic supports.

9.4.2.2 Crossbeams

(1) The influence of the cut outs should be taken into account in the analysis for crossbeams.

NOTE: Where crossbeams are provided with cut outs as given in Figure 9.3, the action effects may be determined with a Vierendeel-model (where the deckplate and a part of the crossbeam below the cut outs are the flanges and the areas between the cut outs are the posts).

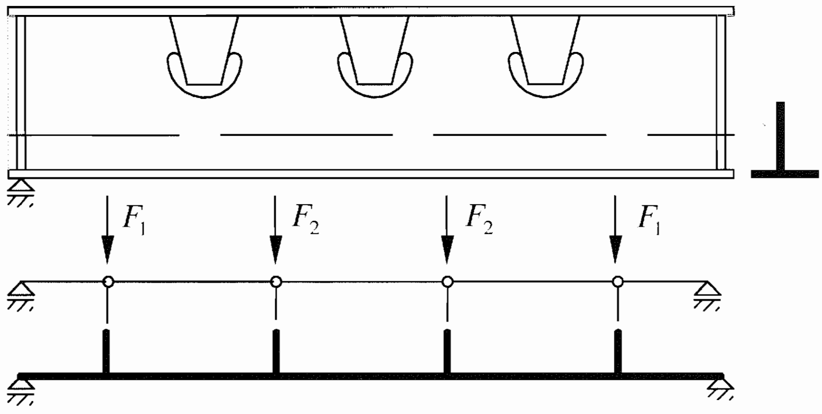


Figure 9.3: Vierendeel-model for a crossbeam

(2) In the analysis of the model for a crossbeam the following should be taken into account:

1. the connections of the crossbeam to the transverse stiffeners of the webs of main girders should form a continuous transverse frame;
2. the contributions of the deformations of components of the Vierendeel-beams due to bending moments, axial forces and shear forces to the overall deformation;
3. the effects of shear between the deckplate and the web of the crossbeam on the direct stresses and shear stresses at the critical section in Figure 9.4;
4. the effects of local introduction of loads from the stiffeners into the web;
5. the shear stresses from the horizontal and vertical shear in the critical section in Figure 9.4.

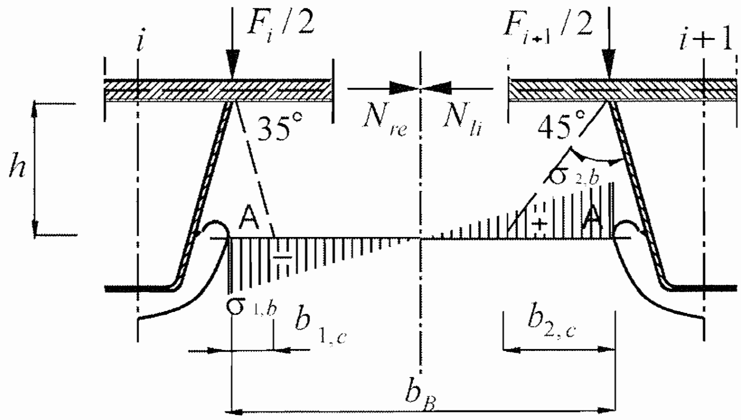


Figure 9.4: Stress distribution at cope hole

(3) The direct stresses in the critical section in Figure 9.4 may be determined as follows:

$$\sigma_1 = \sigma_{1b} + \sigma_{1c} \tag{9.3}$$

$$\sigma_2 = \sigma_{2b} + \sigma_{2c} \tag{9.4}$$

where $-\sigma_{1b} = +\sigma_{2b} = \dfrac{M_{Ed}}{W}$ are the stresses due to bending (9.5)

$\sigma_{1c} = -\dfrac{F_i}{2A_{1c}}$ and $\sigma_{2c} = -\dfrac{F_{i+1}}{2A_{2c}}$ are the compressive stresses due to local load from stiffeners (9.6)

$W = \dfrac{1}{6} t\, b_B^2$

$A_{1c} = b_{1c}\, t$

$A_{2c} = b_{2c}\, t$

V_{Ed} is the horizontal shear force

$M_{Ed} = V_{Ed}\, h$ is the bending moment in the critical section

F_i, F_{i+1} are the loads introduced from the stiffeners

t is the plate thickness of the web

(4) Where no cope holes are provided the stresses at the critical section may be determined using flanges from the webs of the stiffeners with an effective width $b_{eff} = 5\, t_{w,st}$, where $t_{w,st}$ is the plate thickness of the stiffeners.

9.5 Fatigue assessment procedures

9.5.1 Fatigue assessment

(1) The fatigue assessment should be carried out as follows:

$$\gamma_{Ff}\, \Delta\sigma_{E2} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \tag{9.7}$$

and

$$\gamma_{Ff}\, \Delta\tau_{E2} \leq \frac{\Delta\tau_c}{\gamma_{Mf}} \tag{9.8}$$

9.5.2 Damage equivalence factors λ for road bridges

(1) The damage equivalence factor λ for road bridges up to 80m span should be obtained from:

$$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \qquad \text{but } \lambda \leq \lambda_{max} \tag{9.9}$$

where λ_1 is the factor for the damage effect of traffic and depends on the length of the critical influence line or area;

λ_2 is the factor for the traffic volume;

λ_3 is the factor for the design life of the bridge;

λ_4 is the factor for the traffic on other lanes;

λ_{max} is the maximum λ-value taking account of the fatigue limit, see AC1 (7) AC1.

(2) In determining λ_1 the critical length of the influence line or area may be taken as follows:

a) for moments:

- for a simply supported span, the span length L_i;
- for continuous spans in midspan sections, see Figure 9.7, the span length L_i of the span under consideration;
- for continuous spans in support sections, see Figure 9.7, the mean of the two spans L_i and L_j adjacent to that support;
- for cross girders supporting stringers, the sum of the two adjacent spans of the stiffeners carried by the cross girder.

b) for shear for a simply supported span and a continuous span:

- for the support section, see Figure 9.7, the span under consideration L_i;
- for the midspan section, see Figure 9.7, 0,4 × the span under consideration L_i.

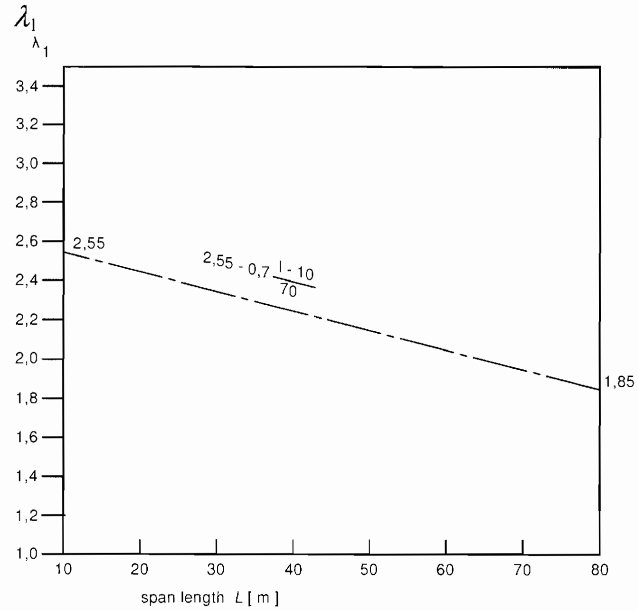
c) for reactions:

- for end support, the span under consideration L_i;
- for intermediate supports, the sum of the two adjacent spans $L_i + L_j$.

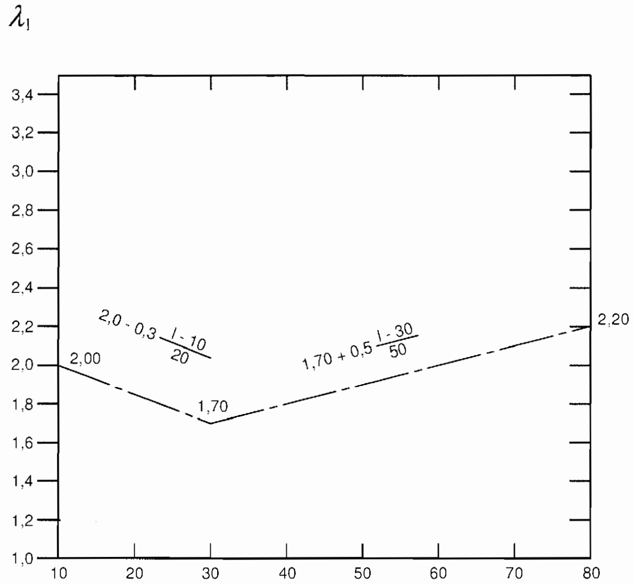
d) for arch bridges:

- AC1) for hangers, twice the distance between hangers; (AC1
- for arch, half the span of the arch.

NOTE: The National Annex may give the relevant values for λ_1. The use of the factors λ_1 in Figure 9.5 is recommended.



span length L [m]
at midspan



span length L [m]
at support

Figure 9.5: λ_1 for moments for road bridges

(3) λ_2 should be calculated as follows:

$$\lambda_2 = \frac{Q_{m1}}{Q_0}\left(\frac{N_{Obs}}{N_0}\right)^{1/5} \tag{9.10}$$

where Q_{m1} is the average gross weight (kN) of the lorries in the slow lane obtained from:

$$Q_{m1} = \left(\frac{\sum n_i Q_i^5}{\sum n_i}\right)^{1/5}$$

$Q_0 = 480$ kN

$N_0 = 0{,}5 \times 10^6$

N_{Obs} is the total number of lorries per year in the slow lane, see 9.2.2(2);

Q_i is the gross weight in kN of the lorry i in the slow lane as specified by the competent authority;

n_i is the number of lorries of gross weight Q_i in the slow lane as specified by the competent authority.

NOTE: The National Annex may give guidance on λ_2.

(4) For given values of Q_{m1} and N_{Obs}, λ_2 may be obtained from Table 9.1.

Table 9.1: λ_2

Q_{m1}	N_{Obs}							
	$0{,}25\times10^6$	$0{,}50\times10^6$	$0{,}75\times10^6$	$1{,}00\times10^6$	$1{,}25\times10^6$	$1{,}50\times10^6$	$1{,}75\times10^6$	$2{,}00\times10^6$
200	0,362	0,417	0,452	0,479	0,500	0,519	0,535	0,550
300	0,544	0,625	0,678	0,712	0,751	0,779	0,803	0,825
400	0,725	0,833	0,904	0,957	1,001	1,038	1,071	1,100
500	0,907	1,042	1,130	1,197	1,251	1,298	1,338	1,374
600	1,088	1,250	1,356	1,436	1,501	1,557	1,606	1,649

(5) λ_3 should be calculated as follows:

$$\lambda_3 = \left(\frac{t_{Ld}}{100}\right)^{1/5} \tag{9.11}$$

where t_{Ld} is the design life of the bridge in years.

Table 9.2: λ_3

Design life in years	50	60	70	80	90	100	120
Factor λ_3	0,871	0,903	0,931	0,956	0,979	1,00	1,037

NOTE: The design life of the bridge t_{Ld} may be specified in the National Annex. The choice of $t_{Ld} = 100$ years is recommended.

(6) λ_4 should be calculated as follows:

$$\lambda_4 = \left[1 + \frac{N_2}{N_1}\left(\frac{\eta_2 Q_{m2}}{\eta_1 Q_{m1}}\right)^5 + \frac{N_3}{N_1}\left(\frac{\eta_3 Q_{m3}}{\eta_1 Q_{m1}}\right)^5 + \dots + \frac{N_k}{N_1}\left(\frac{\eta_k Q_{mk}}{\eta_1 Q_{m1}}\right)^5\right]^{1/5} \tag{9.12}$$

where k is the number of lanes with heavy traffic;

N_j is the number of lorries per year in lane j;

Q_{mj} is the average gross weight of the lorries in lane j;

η_j is the value of the influence line for the internal force that produces the stress range in the middle of lane j to be inserted in equation (9.12) with positive sign.

NOTE: The National Annex may give guidance on λ_4.

(7) The factor λ_{max} should be obtained from the relevant [AC1) stress-range spectrum. (AC1]

NOTE: The National Annex may give the relevant factors λ_{max}. The use of the factors λ_{max} in Figure 9.6 is recommended.

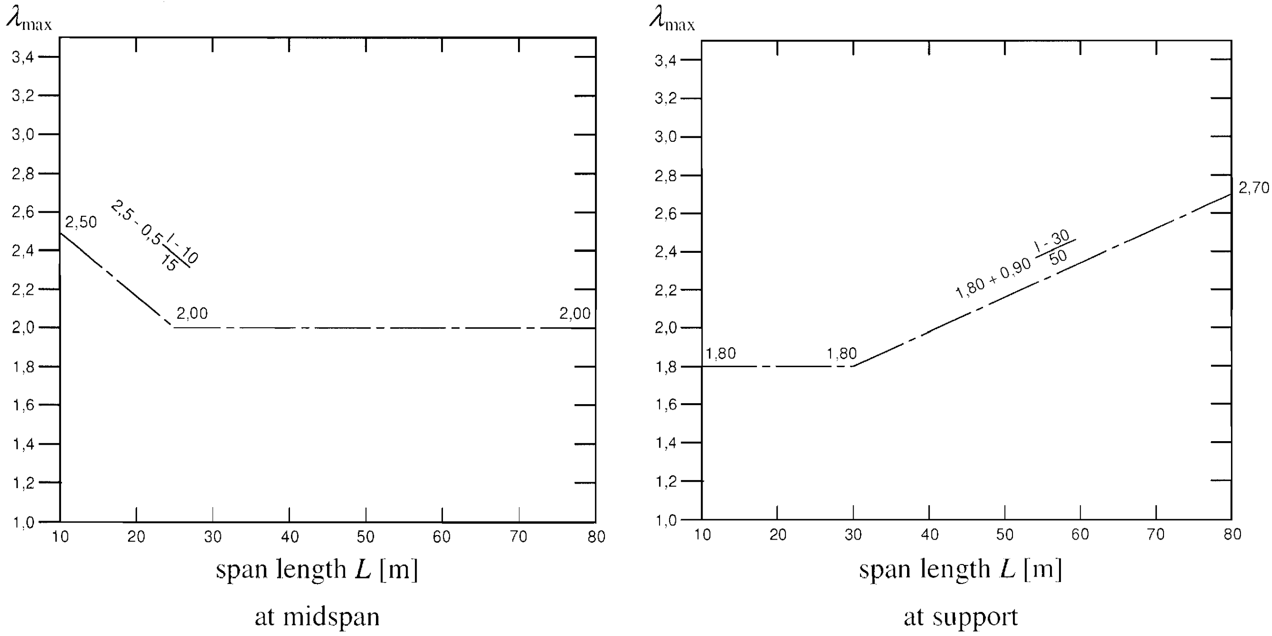


Figure 9.6: λ_{max} for moments for road bridges

9.5.3 Damage equivalence factors λ for railway bridges

(1) The damage equivalence factor λ for railway bridges with a span up to 100 m should be determined as follows:

$$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \qquad \text{but } \lambda \leq \lambda_{max} \qquad (9.13)$$

where λ_1 is the factor for the damage effect of traffic and depends on the length of the influence line;

λ_2 is the factor for the traffic volume;

λ_3 is the factor for the design life of the bridge;

λ_4 is the factor for the structural element is loaded by more than one track;

λ_{max} is the maximum λ value taking account of the fatigue limit, see (9).

(2) λ_1 may be obtained from Table 9.3 and Table 9.4.

NOTE 1: The National Annex may give further guidance for the use of Table 9.3 or Table 9.4.

NOTE 2: The values given in Table 9.3 and Table 9.4 for mixed traffic correspond to the combination of train types given in Annex F of EN 1991-2.

NOTE 3: For lines with train type combinations other than those taken into consideration (specialised lines for example), the National Annex may specify values of λ_1.

Table 9.3: λ_1 for standard rail traffic

L	EC Mix
0,5	1,60
1,0	1,60
1,5	1,60
2,0	1,46
2,5	1,38
3,0	1,35
3,5	1,17
4,0	1,07
4,5	1,02
5,0	1,03
6,0	1,03
7,0	0,97
8,0	0,92
9,0	0,88
10,0	0,85
12,5	0,82
15,0	0,76
17,5	0,70
20,0	0,67
25,0	0,66
30,0	0,65
35,0	0,64
40,0	0,64
45,0	0,64
50,0	0,63
60,0	0,63
70,0	0,62
80,0	0,61
90,0	0,61
100	0,60

Table 9.4: λ_1 for express multiple units and underground and for rail traffic with 25 t axles

	Express multiple units and underground		Rail traffic with 25 t axles
L	Type 9	Type 10	25 t Mix
0,5	0,97	1,00	1,65
1,0	0,97	1,00	1,65
1,5	0,97	1,00	1,65
2,0	0,97	0,99	1,64
2,5	0,95	0,97	1,55
3,0	0,85	0,94	1,51
3,5	0,76	0,85	1,31
4,0	0,65	0,71	1,16
4,5	0,59	0,65	1,08
5,0	0,55	0,62	1,07
6,0	0,58	0,63	1,04
7,0	0,58	0,60	1,02
8,0	0,56	0,60	0,99
9,0	0,56	0,55	0,96
10,0	0,56	0,51	0,93
12,5	0,55	0,47	0,90
15,0	0,50	0,44	0,92
17,5	0,46	0,44	0,73
20,0	0,44	0,43	0,68
25,0	0,40	0,41	0,65
30,0	0,37	0,42	0,64
35,0	0,36	0,44	0,65
40,0	0,35	0,46	0,65
45,0	0,35	0,47	0,65
50,0	0,36	0,48	0,66
60,0	0,39	0,48	0,66
70,0	0,40	0,49	0,66
80,0	0,39	0,49	0,66
90,0	0,39	0,48	0,66
100,0	0,40	0,48	0,66

(4) In determining λ_1 the critical length of the influence line should be taken as follows:

a) for moments:

- for a simply supported span, the span length, L_i;
- for continuous spans in midspan sections, see Figure 9.7, the span length L_i of the span under consideration;
- for continuous spans in support sections, see Figure 9.7, the mean of the two spans L_i and L_j adjacent to that support;
- for cross-girders supporting rail bearers (or stringers), the sum of the two adjacent spans of the rail-bearers (or stringers) immediately adjacent to the cross-girder;
- for a deck plate supported only by cross-girders or cross-ribs (no longitudinal members) and for those supporting cross-members, the length of the influence line for deflection (ignoring any part indicating upward deflection) taking due account of the stiffness of the rails in load distribution. For cross-members spaced not more than 750 mm apart, this may be taken as 2 × cross-member-spacing + 3 m.

b) for shear for both a simply-supported span and a continuous span:

- for the support section, see Figure 9.7, the span under consideration L_i;
- for the midspan section, see Figure 9.7, 0,4 × the span under consideration L_i.

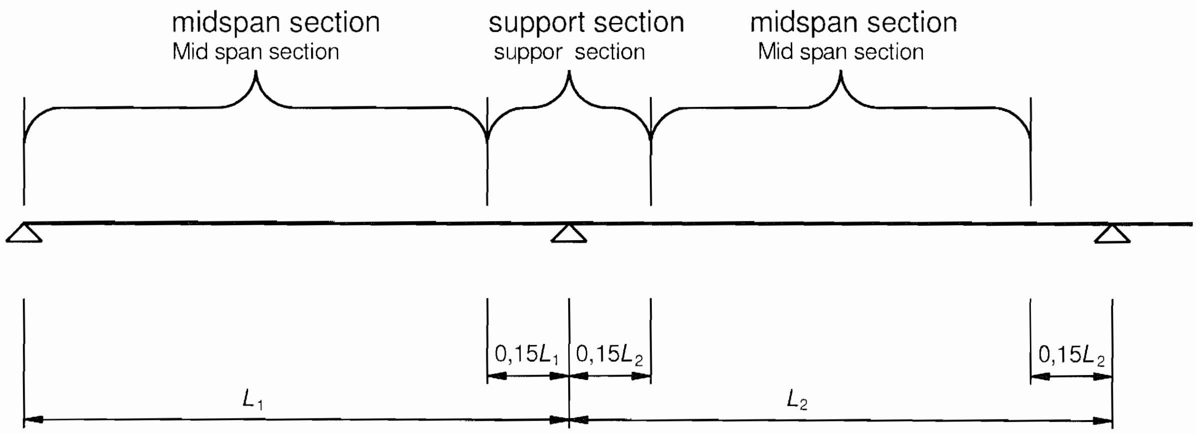


Figure 9.7: Location of midspan or support section

(5) λ_2 should be obtained from Table 9.5.

Table 9.5: λ_2

Traffic per year [10^6 t/track]	5	10	15	20	25	30	35	40	50
λ_2	0,72	0,83	0,90	0,96	1,00	1,04	1,07	1,10	1,15

(6) λ_3 should be obtained from Table 9.6.

Table 9.6: λ_3

Design life [years]	50	60	70	80	90	100	120
λ_3	0,87	0,90	0,93	0,96	0,98	1,00	1,04

(7) λ_4 should be obtained from Table 9.7.

Table 9.7: λ_4

$\Delta\sigma_1/\Delta\sigma_{1+2}$	1,00	0,90	0,80	0,70	0,60	0,50
λ_4	1,00	0,91	0,84	0,77	0,72	0,71
$\Delta\sigma_1$ is the stress range at the section to be checked due to load model 71 on one track;						
$\Delta\sigma_{1+2}$ is the stress range at the same section due to load model 71 according to EN 1991-2 on any two tracks.						

NOTE: Table 9.7 is only valid if $\Delta\sigma_1$ and $\Delta\sigma_{1+2}$ have the same sign.

(8) The values of λ_4 in Table 9.7 assume that 12 % of the total traffic crosses the bridge whilst there is traffic on the other track. If the percentage of traffic crossing the bridge is different, λ_4 should be taken as follows:

$$\lambda_4 = \sqrt[5]{n + [1-n][a^5 + (1-a)^5]} \qquad (9.14)$$

where $a = \Delta\sigma_1 / \Delta\sigma_{1+2}$

n is the percentage of traffic.

(9) The value of λ should not exceed λ_{max} given by:

$$\lambda_{max} = 1{,}4 \qquad (9.15)$$

9.5.4 Combination of damage from local and global stress ranges

(1) Where the stress verification in a member is due to the combined effects of flexure of the bridge (global effects) and flexure of the internal elements (local effects), the combined effects $\Delta\sigma_{E2}$ should be as follows:

$$\Delta\sigma_{E2} = \lambda_{loc} \times \Phi_{loc} \times \Delta\sigma_{loc} + \lambda_{glo} \times \Phi_{glo} \times \Delta\sigma_{glo} \qquad (9.16)$$

where the suffix "loc" refers to local effects and "glo" refers to global effects.

9.6 Fatigue strength

(1) EN 1993-1-9 should be used for the fatigue strength assessment of bridges.

NOTE 1: The National Annex may exclude particular details in EN 1993-1-9 from the design of bridges.

NOTE 2: The National Annex may give supplementary guidance for the fatigue of deck plates.

(2) For the critical regions of steel decks, the fatigue categories given in Table 9.8 may be used.

Table 9.8: Detail categories for fatigue assessments

Critical region	Detail	Detail to EN 1993-1-9	Detail category
1	Deckplate stressed longitudinally at transverse fillet welds, see Figure 9.1	Table 8.4 detail 8	71
2	Deckplate stressed longitudinally at welded stringer-to-deckplate connection, see Figure 9.1	Table 8.2 detail 6	100
		Table 8.3 detail 9	80
3	Hollow section stiffener at stiffener-crossbeam connection, see Figure 9.1	Table 8.8 detail 1	80
4	Splice of stiffeners with splice plates and metallic backing strips, see Figure 9.2	Table 8.8 detail 4	71
5	AC1) Free edges of cope holes in webs of crossbeams, see Figure 9.4 (AC1	Table 8.8 detail 6	112

9.7 Post weld treatment

(1) Where appropriate, weld improvement techniques such as weld toe grinding, TIG remelting of weld toe region, hammer peening, shot peening, may be used to improve the fatigue life of connections.

NOTE: The National Annex may give provisions for post weld treatment.

10 Design assisted by testing

10.1 General

(1) Design assisted by testing should be in accordance with EN 1990, supplemented by the additional provisions given in 10.2 and 10.3.

10.2 Types of tests

(1) Tests may be carried out as follows:

a) to determine the ultimate resistance or serviceability properties of structural parts, e.g. tests to develop standardised temporary bridge systems;

b) to obtain specific material properties, e.g. soil testing in situ or in the laboratory, testing of new materials for coating;

c) to reduce uncertainties in parameters in load or resistance models, e.g. wind tunnel testing, testing of full size prototypes, testing of small scale models;

d) to check the quality of the delivered products or the consistency of the production characteristics, e.g. tests of cables or sockets;

e) to take account of actual conditions experienced, e.g. for measurements of frequencies or damping;

f) to check the behaviour of the actual structure or of structural elements after completion, e.g. proof load tests at the ultimate or serviceability limit states.

(2) For test types a), b) and c), the design values should be obtained from the test results, if these are available at the time of design.

(3) For test types d), e) and f) or situations where the test results are not available at the time of design, the design values should be taken as those that are expected to satisfy the acceptance criteria at a later stage.

10.3 Verification of aerodynamic effects on bridges by testing

(1) Testing should be used to verify the design of a bridge under the effects of wind where the calculation or the use of established results do not to provide sufficient assurance of the structural safety during either the erection stage or the service life.

(2) Testing should be used to determine:

a) the overall wind environment at the bridge site and at the local wind recording station;

b) the quasi-static drag and lift forces and twisting moments on a bridge or its components resulting from the flow of wind past them;

c) the amplitude of oscillation of the bridge or its components due to vortex shedding from alternate sides of the bridge or its components in the wind flow (limited amplitude response);

d) the wind speed at which the bridge or its components may be liable to a divergent amplitude response due to galloping, stall flutter, classical flutter, rain-wind-induced vibration, non-oscillatory divergence, etc;

e) the response of the bridge or its element due to the turbulence in the natural wind;

f) the inherent damping of the structure.

(3) 10.3(2)a) to e) above should be carried out in a wind tunnel. Where a bridge is subject to wind tunnel testing, the models should accurately simulate the external cross-sectional details including non-structural fittings, such as parapets. A representative range of natural frequencies and damping, appropriate to the predicted modes of vibration of the bridge, should also be simulated. Due consideration should be given to the influence of turbulence and to the effect of wind which is inclined to the horizontal.

(4) Any potential changes in cross section (including icing or rivulets of water on a cable) should be taken into account when testing.

NOTE: The structural damping may be determined by mechanically exciting the bridge (using reciprocating machinery, out of balance rotating machinery, rockers or similar devices). The value of damping required can be determined from the energy required to generate a particular amplitude of oscillation or the decay of oscillation after the excitation has ceased.

Annex A [informative] – Technical specifications for bearings

A.1 Scope

(1) This annex gives guidance for preparation of technical specifications for bearings, that comply with EN 1337. The following bearings are not covered:

a) bearings that transmit moments as a primary function;

b) bearings that resist uplift;

c) bearings for moving bridges;

d) concrete hinges;

e) seismic devices.

NOTE 1: This annex is intended to be transferred to EN 1990 – Basis of structural design.

NOTE 2: Fixed bearings prevent movements but other bearings such as guided bearings allow movements in one direction while free bearings allow movements in all directions.

NOTE 3: Detailed information on bearings may be obtained from the following parts of EN 1337:

Part 1: General

- General design rules

Part 2: Sliding elements

- AC1) Vertical bearing resistance (AC1
- Reaction forces due to friction
- Translation capability
- Eccentricity

Part 3: Elastomeric bearings

- AC1) Vertical bearing resistance (AC1
- Reaction forces due to horizontal deformations
- Reaction moments due to rotation about the horizontal axes
- Eccentricity

Part 4: Roller bearings

- AC1) Vertical bearing resistance (AC1
- Reaction forces due to "rolling" friction
- Reaction moment in vertical plane with roller axis
- Horizontal bearing capacity due to friction in direction of roller axis
- Rotation about roller axis
- Eccentricity of roller with respect to top plate and bottom plate 0,5 times the relative eccentricity between the main structures

Part 5: Pot bearings

- AC1) Vertical bearing resistance (AC1
- Reaction moment in vertical plane
- Wear of seal
- Rotation capacity

Part 6: Rocker bearings

- (AC1) Vertical bearing resistance (AC1)
- Horizontal bearing capacity due to friction
- Rotation capacity about one axis

Part 7: Spherical and cylindrical PTFE bearings

- (AC1) Vertical bearing resistance (AC1)
- Reaction moment(s) due to friction
- Rotation capacity about all (spherical) axes or one (cylindrical) axis

Part 8: Guided bearings and restraint bearings

- Restraint of movements in one or more directions

Part 9: Protection

Part 10: Inspection and maintenance

Part 11: Transport, Storage and installation

(2) For technical specifications for bearings including vertical and horizontal forces, translational and rotational movements and other geometrical and performance characteristics, see A.3.1 (3).

A.2 Symbols

(1) Symbols for the most common types of bearings may be taken from EN 1337-1, Table 1.

A.3 General

A.3.1 Bearing layout

(1) The bearing layout should be designed to permit the specified movement of a structure with the minimum possible resistance to such movements.

(2) The arrangement of bearings for a structure should be considered in conjunction with the design of the structure as a whole. The forces and movements in bearings should be given to the bearing manufacturer to ensure that the bearings provided meet the requirements.

(3) A drawing showing the bearing layout should include the following:

a) a simplified general arrangement of the bridge showing the bearings in plan;

b) details at the bearing location (e.g. recess and reinforcement) ;

c) a clear indication of the type of bearing at each location;

d) a table giving the detailed requirements for each bearing;

e) bedding and fixing details.

(4) Bearings should not normally be expected to resist moments due to rotational movement. Where such rotational movement is present provision should be made to accommodate it by using the bearing itself or within the structure. Where bearings are required to resist rotational movement an analysis should be carried to ensure that the bearings will not be affected adversely, see A.3.2.

(5) Uplift may cause excessive wear in bearings if such conditions occur frequently. Where uplift is unavoidable prestressing may be used to provide the necessary additional vertical force.

(6) Bearings and supports should be designed in such a way that they can be inspected, maintained and replaced if necessary.

NOTE 1: For inspection purposes bearings should be provided with movement indicators with markings showing the maximum allowable movements.

NOTE 2: A clearance of not more than 10 mm should be provided for resetting or replacement of bearings or parts of bearings during jacking of the structure.

(7) If presetting is required it should be carried out at the factory wherever possible. If adjustment on site is unavoidable it should be carried out in accordance with the manufacturers' detailed instructions.

A.3.2 Effects of continuity of deformation

(1) For line rocker and single roller bearings, the full implications of uneven pressure along the length of the roller or rocker should be taken into account in the design of the structure and the bearing. Particular care should be taken in the design of the following:

a) structures curved in plan;

b) structures with slender piers;

c) structures without transverse beams;

d) structures with transverse beams where the line rocker or single roller could effectively act as a built-in support for the transverse beam;

e) structures with a transverse temperature gradient.

A.3.3 Anchorage of bearings

(1)P Anchorages of bridge bearings shall be designed at the ultimate limit state. Where the position of a bearing or part of a bearing is retained either completely or partially by friction its safety against sliding shall be checked in accordance with the following

$$V_{Ed} \leq V_{Rd} \tag{A.1}$$

where V_{Ed} is the design value of the shear force acting at the bridge bearing

$$V_{Rd} = \frac{\mu_K}{\gamma_\mu} N_{Ed} + V_{pd}$$

N_{Ed} is the minimum design force acting normal to the joint in conjunction with V_{Ed};

V_{pd} is the design value of shear resistance of any fixing device in accordance with the Eurocodes;

μ_K is the characteristic value of the friction coefficient, see Table A.1;

γ_μ is the partial factor for friction.

NOTE: The value for γ_μ may be given in the National Annex. The following values are recommended.

$\gamma_\mu = 2{,}0$ for steel on steel

$\gamma_\mu = 1{,}2$ for steel on concrete

Table A.1: Characteristic values of the friction coefficient μ_K

Surface treatment of steel components	Steel on steel	Steel on concrete
Uncoated and free from grease Metal-sprayed Coated with fully hardened zinc silicate	0,4	0,6
Other treatment	From test	From test

(2) For dynamically loaded structures the value of N_{Ed} should be determined taking into account any dynamic variations in traffic loads.

(3) For railway bridges and structures subjected to seismic situations friction should not be taken into account ($N_{Ed} = 0$).

(4) Where holding down bolts or other similar devices are used to provide some of the resistance to horizontal movement, it should be demonstrated that this resistance is provided before any movement can take place. If bolts are provided in holes with normal tolerances, movement will inevitably take place before the full resistance to movement is achieved. This is unacceptable in service conditions.

A.3.4 Conditions of installation

(1) Conditions of installation taking into account the construction sequence and other time dependent effects should be determined and agreed with the manufacturer.

NOTE: In view of the difficulties of predicting conditions on site at the time of installation the design of bearings should be based on a number of alternative assumptions, see A.4.2.

A.3.5 Bearing clearances

(1) Where the bearings are designed to resist horizontal forces, some movements will take place before clearances are taken up.

(2) The total clearance between extremes of movements may be up to 2 mm unless otherwise specified or agreed with the manufacturer.

(3) Clearance should not be taken into account when allowing for horizontal movements unless it can be shown that these movements will be permanently available in the correct direction.

(4) If more than one bearing is required to resist horizontal forces, the bearings and their supports should be designed to ensure that an adverse distribution of clearance will not prevent this happening. They should also be designed to accommodate the sharing of the load between the bearings caused by any distribution of clearance.

A.3.6 Resistance of bearings to rolling and sliding

(1) The resistance to movement of the various types of bearings may be calculated in accordance with EN 1337.

NOTE 1: The calculation needs to allow for the most adverse combination of the permitted variation in material properties, environmental conditions and manufacturing and installation tolerances.

NOTE 2: The properties of some materials (e.g. wear or friction coefficient of PTFE or stress-strain behaviour of elastomers) are only valid for the specified temperature range and the movement speeds that normally occur in structures. They are only valid when the bearings are adequately maintained and protected from harmful substances.

NOTE 3: The actual resistance to movement is likely to be considerably less than the calculated maximum. Therefore, it should not be considered in the design when favourable except as given in (2) below.

(2) Where a number of bearings of equal type are arranged in such a way that the adverse forces, resulting from the resistance to movement by some bearings are partly relieved by the forces resulting from the resistance to movement by others, the respective coefficients of friction μ_a and μ_r should be calculated as follows:

$$\mu_a = 0{,}5\ \mu_{max}\ (1 + \alpha) \tag{A.2}$$

$$\mu_r = 0{,}5\ \mu_{max}\ (1 - \alpha) \tag{A.3}$$

where μ_a is the adverse coefficient of friction;

μ_r is the relieving coefficient of friction;

μ_{max} is the maximum coefficient of friction for the bearing as given in the relevant Parts of EN 1337;

α is a factor dependent on the type of bearing and the number of bearings which are exerting either an adverse or relieving force as appropriate.

NOTE: The value for α may be chosen in the National Annex. Recommended values are given in Table A.2.

Table A.2: Factors α

n	α
≤ 4	1
$4 < n < 10$	$\dfrac{16-n}{12}$
≥ 10	0,5

(3) Clause (2) may also be applied to elastomeric bearings which come from different manufacturers. In such a case the coefficients of friction in equation (A.2) and (A.3) may be substituted by the respective shear moduli.

A.4 Preparation of the bearing schedule

A.4.1 General

(1) The bearing schedule should ensure that bearings are designed and constructed in such a way that under the influence of all possible actions, unfavourable effects of the bearing on the structure are avoided.

(2) The bearing schedule should contain:

- a list of forces on the bearings from each action;
- a list of movements of the bearings from each action;
- other performance characteristics of the bearings.

NOTE 1: Forces and movements from the various actions during construction are to be appropriate to the construction and inspection scheme including time dependent effects.

NOTE 2: Forces and movements from variable actions are to be given extreme minimum and maximum values corresponding to the relevant load positions

NOTE 3: All forces and movements from actions other than temperature are to be given for a specified temperature T_0. The effects of temperature need to be determined in such a way that the effects of deviation from the specified temperature T_0 can be identified.

(3) For structures with elastic behaviour, all forces and movements should be based on characteristic values of actions. The relevant partial factors and combination rules should be applied at serviceability, ultimate or durability limit states.

NOTE 1: Guidance for a bearing schedule with characteristic values of bearing reactions and displacements is given in Table A.3. Design values representing the technical specifications for bearing are to be derived from this table.

NOTE 2: Normally the most adverse combination of action effects is sufficient for the design of bearings, see Table A.3. In special cases greater economy may be achieved by considering the actual coexistent values of action effects.

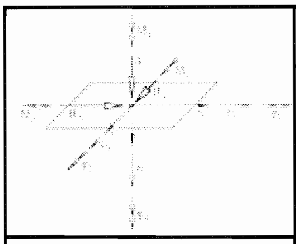
(4) For structures in which the deformations are significant for action effects second order analysis may be performed in two stages:

a) for the actions during the various construction phases up to the attainment of the final form of the structure that are required after construction for a specified temperature;

b) for all variable actions imposed on the final form of the structure.

NOTE: In general there is a requirement for the final geometrical form of the bridge (including its bearings) to be specified for a particular temperature after completion of construction. This is used as a reference for determining the necessary measures during construction and also for determining forces and movements from variable actions during service taking into account any uncertainties.

Table A.3: Typical bearing schedule

				Bearing reactions and displacements										Bearing No.	
			reaction *)	max A	min A	max H_x	min H_x	max H_y	min H_y	max M_z	min M_z	max M_x	min M_x	max M_y	min M_y
				[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm]	[kNm]	[kNm]
			displacement *)	max w	min w	max e_x	min e_x	max e_y	min e_y	max f_z	min f_z	max f_x	min f_x	max f_y	min f_y
actions (characteristic values)				[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mrad]	[mrad]	[mrad]	[mrad]	[mrad]	[mrad]
1.1	permanent G, P	self weight													
1.2		dead load													
1.3		prestressing													
1.4		creep and shrinkage													
2.1	variable Q	traffic loads													
2.2		special vehicles	and/or 2.1												
2.3		centrifugal force													
2.4		braking and acceleration forces													
2.5		nosing forces													
2.6		footpath loading													
2.7		wind on structure	w/o 2.1 to 2.6/or 2.8												
2.8		wind on structure and traffic	or 2.7												
2.9		temperature													
2.10		vertical temperature gradient													
2.11		horizontal temperature gradient													
2.12		settlement substructure													
2.13		restraint / friction force													
3.1	seismic	non collapse rupture (ULS)													
3.2		minimisation of damage (SLS)													
4.1	accidental A	derailment													
4.2		collision													
4.3		rupture of overhead line													
5.1	combinations														
5.2															
5.3															
5.4															
5.5															
...															
*) delete if not applicable								given by the designer of the bridge						given by the producer of the bearing	

This list comprises all reactions and movements in the final stage. When the bearings are installed during erection, they should be readjusted after reaching the final stage and reactions and movements exceeding those of the final stage should be give separately.

A.4.2 Determination of design values of actions on the bearings and movements of the bearings

A.4.2.1 General

(1) In determining the actions on bearings and their movements the following reference situation should be recorded on the drawings:

a) Final geometrical form of the completed bridge for the reference temperature T_0;

b) The locations of the fixed bearings and the sliding bearings at the time of installation for the reference temperature T_0;

c) for elastomeric bearings, the position and movements of the bearings at their location should conform to the assumptions made for the reference temperature T_0;

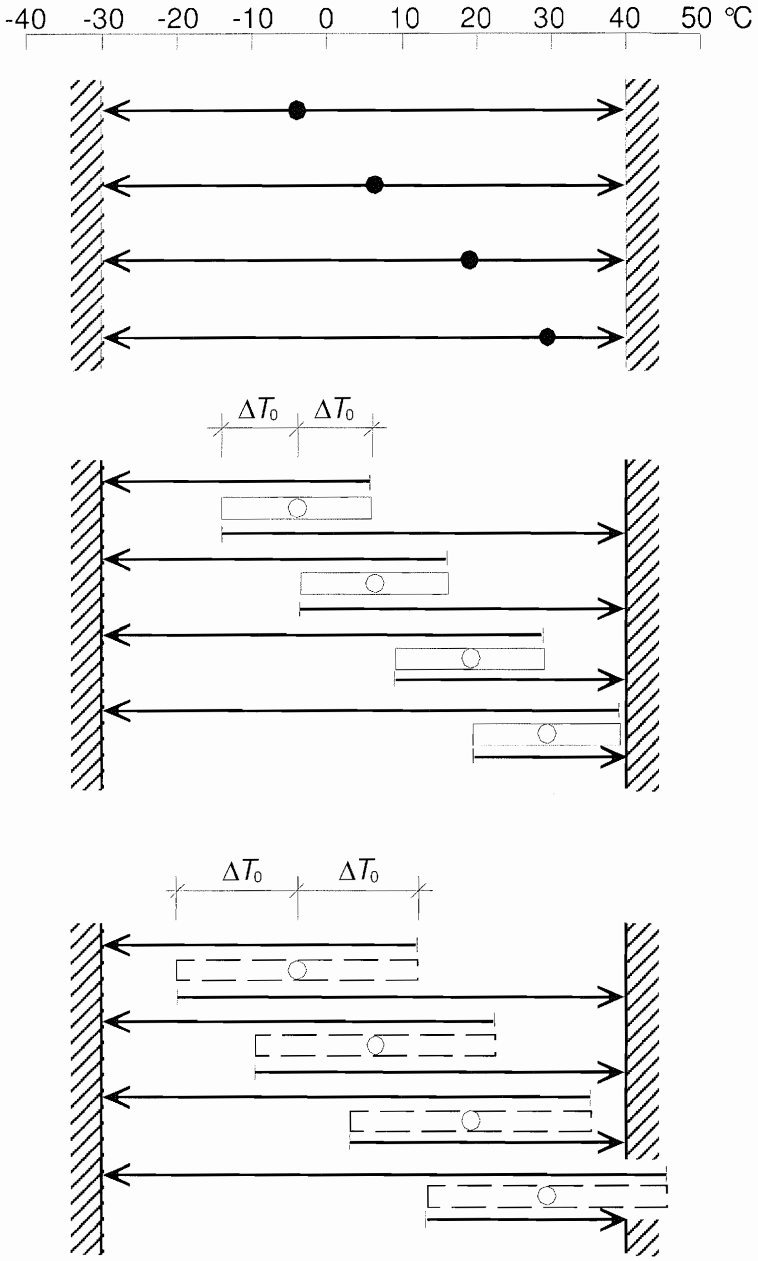
d) any uncertainty of position of the bearings at the reference temperature T_0, that may give rise to enlarged movements or restraints to such movements, is included in the assumptions for the design values of the reference temperature T_0 and, consequently, for the design values of the temperature differences ΔT_d^*.

(2) The uncertainty of position of the sliding bearings in relation to the position of the fixed bearings, or in case of elastomeric bearings in relation to the neutral point of movement for both permanent actions at the time of completion of the bridge, and the given reference temperature T_0 depends on:

a) the method of installing the bearings;

b) the mean temperature of the bridge when the bearing are installed;

c) the accuracy of measurement of the mean temperature of the bridge, see Figure A.1.



Case 1
Position of sliding bearings after final connection to the fixed bearings with accurate measurements of temperature of the structure

Case 2
Position of bearings without accurate measurement of the temperature of the structure and without correction of the position when the final connection to the fixed bearings is made

Case 3
As case 2 but with one or more changes of location of the fixed bearings

sum of both movements = total movement from temperature difference

● mean temperature of structure as measured

○ estimated mean temperature of structure

error in estimation of mean temperature plus uncertainty from 1 or more changes of location of fixed bearing

error in estimation of mean temperature

realistic limits of temperature for the structure

Figure A.1: Determination of ΔT_0 to take uncertainties of position of bearings into account

NOTE: The National Annex may give guidance on temperature measurements.

(3) The uncertainty of the position of sliding bearings should be taken into account by taking an appropriate upper value T_{0max} and a lower value T_{0min} for the installation. These should be taken as:

$$T_{omax} = T_0 + \Delta T_0 \qquad (A.4)$$

$$T_{omin} = T_0 - \Delta T_0 \qquad (A.5)$$

NOTE: ΔT_0 may be specified in the National Annex. Numerical values of ΔT_0 for steel bridges as given in Table A.4 are recommended.

Table A.4: Numerical values for ΔT_0

Case	Installation of bearings	ΔT_0 [°C]
1	Installation with measured temperature and with correction by resetting	0
2	Installation with estimated temperature and without correction by resetting with bridge set at $T_0 \pm 10$ °C	15
3	Installation with estimated temperature and without correction by resetting and also one or more changes in the position of the fixed bearing	30

(4) The design values of the temperature difference ΔT_d^* including any uncertainty of the position of the bearings should be determined from

$$\Delta T_d^* = \Delta T_K + \Delta T_\gamma + \Delta T_0 \qquad (A.6)$$

where ΔT_K is the characteristic value of the temperature difference in the bridge according to EN 1991-1-5 relative to the mid point of the temperature range;

ΔT_γ is the additional safety term to allow for the temperature difference in the bridge;

ΔT_0 is the safety term to take into account the uncertainty of the position of the bearing at the reference temperature.

NOTE 1: The National Annex may specify ΔT_γ and ΔT_0.

NOTE 2: A numerical example for determining ΔT_d^* for case 2 in Table A.4 is:

$T_{Kmin} = -25$°C

$T_{Kmax} = +45$°C

$\Delta T_K = \pm 35$ °C

$T_0 = +10$ °C

$\Delta T_0 = \pm 15$ °C

$\Delta T_\gamma = \pm 5$ °C

$\Delta T_d^* = 35 + 5 + 15 = \pm 55$ °C

NOTE 3: In using ΔT_d^* for bearings with sliding elements or rollers and for elastomeric bearings the design criteria should be appropriate to ultimate limit states and not to serviceability limit states.

(5) Where actions on bearings and their movements are obtained from a non linear global analysis of the structure (with the bearings being structural components) and incremental calculations are required, the design value of the temperature difference ΔT_d^* may be expressed in terms of:

$$\Delta T_d^* = \gamma_T \, \Delta T_K \tag{A.7}$$

where γ_T is the partial factor for the temperature difference.

NOTE: In the case of the example given in NOTE 2 of A.4.2.1(4) γ_T would take the following values:

case 1 in Table A.4 $\gamma_T = \frac{40}{35} = 1{,}15$

case 2 in Table A.4 $\gamma_T = \frac{55}{35} = 1{,}60$

case 3 in Table A.4 $\gamma_T = \frac{70}{35} = 2{,}00$

(6) For determining the design values of actions on bearings and their movements, the relevant loading combination for the persistent, transient and accidental load combinations should be taken into account.

A.4.2.2 Actions for persistent design situations

(1) Persistent design situations should apply to the bridge after its construction with the required form under permanent actions at the reference temperature T_0.

NOTE: For construction see A.4.2.3.1

(2) Where time dependent actions have to be considered these should be applicable only after construction.

(3) The characteristic values of the actions may be taken from the Eurocodes listed in Table A.5, see also Table A.3.

Table A.5: Characteristic values of actions

No.	Action	Eurocode
01	reference temperature T_0	EN 1991-1-5, Annex A
02	temperature difference ΔT_0	
1.4	creep $\varepsilon_{K\phi_K}$ for $\phi_K = 1{,}35\ \phi_m$	EN 1992-1
	shrinkage $\varepsilon_{SK} = 1{,}6\ \varepsilon_{sm}$	EN 1992-1
2.1	traffic loads	EN 1991-2
2.2	special vehicles	EN 1991-2
2.3	centrifugal forces	EN 1991-2
2.4	brake and acceleration forces	EN 1991-2
2.5	nosing forces	EN 1991-2
2.6	foot path loading	EN 1991-2
2.7	wind on structures	EN 1991-1-4
2.8	wind on structures and traffic	EN 1991-2
2.9	temperature	EN 1991-1-5 6.13 and 6.15
2.10	vertical temperature gradient	EN 1991-1-5 6.14 and 6.15
2.11	horizontal temperature gradient	EN 1991-1-5 6.14 and 6.2
2.12	settlement of substructure	EN 1997-1
2.13	restraint, friction forces	EN 1337

(4) For the combination of actions see A.4.2.7.

A.4.2.3 Actions for transient design situations

A.4.2.3.1 Design situations during construction

(1) Where bearings are installed before the construction is completed, all relevant construction phases after the installation of the bearings including any changes of the boundary conditions of the system and all actions during construction should be taken into account in the calculation of movements.

(2) Time dependent actions that develop during the construction phase should be taken into account.

(3) The form of the bridge required at the time of installation of the bearings may be determined from the form required for the bridge after construction at the reference temperature T_0.

(4) The characteristic values of actions may be taken from the Eurocodes listed in Table A.6, see also Table A.3.

Table A.6: Characteristic values of actions

No.	Action	Eurocode
01	reference temperature T_0	EN 1991-1-5 Annex A
02	temperature difference ΔT_0	
1.1	self weight	EN 1991-1-7
1.2	dead load	EN 1991-1-7
1.3	prestressing	
1.4	creep	EN 1992-1
	shrinkage	EN 1992-1
2.2	erection loads	EN 1991-1-7
2.6	variable loads	EN 1991-1-7
2.7	wind on structure	EN 1991-1-4
2.8	wind during works	EN 1991-1-4
2.9	temperature	EN 1991-1-5
2.10	vertical temperature gradient	EN 1991-1-5
2.11	horizontal temperature gradient	EN 1991-1-5
2.12	settlement of substructure	EN 1997-1
2.13	restraint, friction forces	EN 1337

(5) During launching of bridge girders friction forces, effects of the longitudinal slope of the bridge and sway of the piers should be taken into account.

(6) For the combination of actions, see A.4.2.7.

A.4.2.3.2 Replacement of bearings and other transient design situations

(1) For transient design situations, the representative values of actions may be reduced in accordance with the limited duration of the situation.

NOTE: For transient design situations for traffic see also EN 1991-2.

(2) For the combination of actions see A.4.2.7.

A.4.2.4 Actions for accidental design situations

(1) Accidental design situations may be caused by a number of factors including the following:

- failure of auxiliary devices during launching of a bridge;
- failure of the bearing;
- failure of the foundation or pier.

(2) For actions arising from the above failures or for other accidental situations without defined causes, the movements and displacements of the bridge should be limited by suitable stops at the abutments or on the piers in such a way that damages are limited and slippages of the bridge or piers are prevented.

NOTE: The National Annex may give further guidance.

(3) For the design of accidental design situations see EN 1992 to EN 1999.

(4) For the combination of actions see A.4.2.7.

A.4.2.5 Seismic design situations

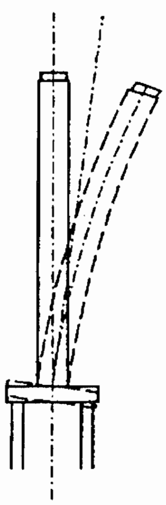
(1) For seismic design situations to determine actions and movements of bearings see EN 1998-1 and EN 1998-2.

(2) For the combination of actions see A.4.2.7.

A.4.2.6 Analysis models for determining the movements of bearings

(1) Where the deformation of the foundation or the piers or the bearings has a significant influence on the forces on bearings or the movements of bearings, these elements should be included in the analysis model.

(2) For linear behaviour the elastic horizontal stiffness of the foundations, piers and bearings may be modelled as individual springs, which may be combined to a global spring stiffness at the location of a bearing for the calculation of the movements and restraints to movements for the various actions, see Figure A.2.



spring model

$K_{\text{foundation}}$ K_{pier} K_{bearing}

spring stiffness K [MN/m]

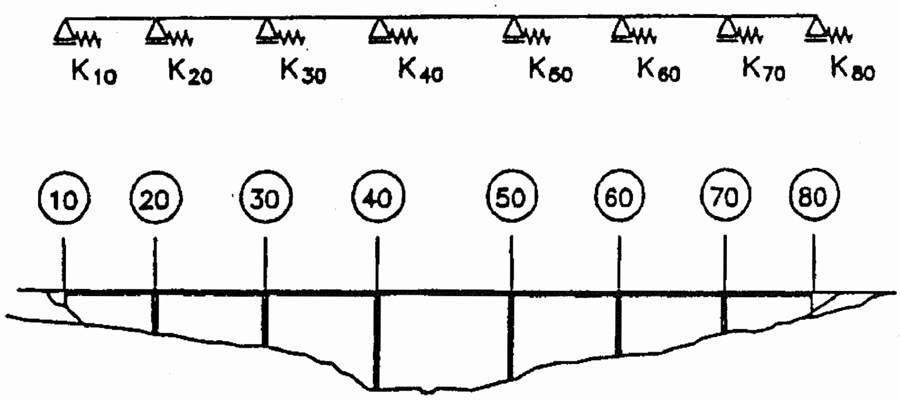
$$\frac{1}{K_{total}} = \frac{1}{K_{foundation}} + \frac{1}{K_{pier}} + \frac{1}{K_{bearing}}$$

displacements of springs v [m/MN]

$$v_{total} = v_{foundation} + v_{pier} + v_{bearing}$$

Figure A.2: Global spring stiffness of pier

(3) The global spring stiffness from all of the pier stiffness in the longitudinal direction of the bridge may be determined from the sum of all the stiffness of the piers, see Figure A.3.



total spring stiffness K [MN/m]

$$K_{total} = K_{10} + K_{20} + K_{30} + K_{40} + K_{50} + K_{60} + K_{70} + K_{80}$$

Figure A.3: Horizontal spring stiffness from the piers

(4) The effects of eccentricity of springs on the distribution of forces should be taken into account.

A.4.2.7 Combinations of actions

(1) For the combination of actions to determine the design values of forces on bearings and movements of bearings in persistent and transient design situations see 6.4.3.2 of EN 1990.

(2) For the partial factors γ_G, γ_P and γ_Q for permanent and variable actions, see Annex A2 of EN 1090.

(3) The following procedure may be used where bearings are installed before the construction of the bridge is completed and where the movements of the bearings are checked during construction by measurements:

1. Actions on bearings and movements should be determined for all relevant construction phases in accordance with A.4.2.3.1. For the characteristic combination of actions 6.5.3(2) of EN 1990 should be used. When second order analysis is used the deformation calculated should be based on the initial form of the structure (form as fabricated without stresses at the reference temperature T_0). A comparison of the measured values and the values as calculated should be recorded and corrections undertaken where appropriate.

   Ultimate limit state verifications for the bearings and the bridge structure at the points of load introduction from the bearings should follow A.4.2.7(1) and A.4.2.7(2) with movements of bearings calculated for the characteristic combination of actions.

2. The calculation of forces on bearings and movements for design values of variable actions that occur after the completion of the bridge should be based on the geometrical form of the bridge and the location of the bearings as required and checked after construction of the bridge at the reference temperature T_0.

   When second order analysis is used, the γ-factors for permanent actions in combination with the action effects from permanent actions should be applied to the required final form of the bridge.

(4) Ultimate limit state verifications for the bearings and the bridge support at the points of load introduction from the bearings should be carried out for the combination of actions in accordance with 6.4.3.2 of EN 1990. Any eccentricity for loads should be obtained from the calculation in A.4.2.7(3).

A.4.3 Determination of the position of bearings at the reference temperature T_0

(1) The installation temperature of the bearing should be such that the temperature expansion and contraction are not markedly different.

(2) Deformation due to creep and shrinkage may be considered to be equivalent to an additional thermal contraction (cooling down).

A.5 Supplementary rules for particular types of bearings

A.5.1 Sliding bearings

(1) The load introduction to the bearing should be proportioned in such a way that the deformation limits of the backing plates of sliding elements see EN 1337-2, 6.9 are not exceeded.

A.5.2 Elastomeric bearings

(1) Forces, moments and deformations exerted on the structure from elastomeric bearings can be determined using the stiffness parameters given in EN 1337-3, 5.3.3.7.

A.5.3 Roller bearings

(1) The eccentricity due to the relative movement of top and bottom roller plates may be increased by eccentricities due to the roller friction and from rotational elements in case of multiple rollers.

(2) For eccentricity in the transverse direction see A.3.2 and A.4.3(2).

A.5.4 Pot bearings

(1) The following procedure may be used to determine the relevant class of accumulated slide path of internal seal systems unless otherwise specified.

NOTE: The class of accumulated slide path of internal seal is related to testing for durability.

(2) It should be verified that

$$S_d \leq S_T \tag{A.8}$$

where S_d is the required accumulated slide path due to variable loads

S_T is the accumulated slide path capacity in accordance with EN 1337-5, 5.4 or from testing according to Annex E of EN 1337-5.

(3) S_d may be determined from

$$S_d = \frac{D}{2c} \sum_i n_i \Delta\phi_{2i} \tag{A.9}$$

where n_i is the number of load events associated with the effects $\Delta\phi_{2i}$;

$\Delta\phi_2 = \phi_{2max} - \phi_{2min}$ is the range of rotation angles from extreme positions of the characteristic loads;

D is the internal diameter of pot in mm;

c is a factor to correct for the difference between the constant amplitude slide path used in tests and the actual effects of variable amplitude movements.

NOTE: AC1) *Text deleted* (AC1 c may be taken as $c = 5$, unless otherwise specified.

(4) For restraint moments due to rotation of elastomeric pad and internal seal friction, see EN 1337-5, 6.13.

A.5.5 Rocker bearings

(1) For line and point rocker rotational eccentricities, see EN 1337-6, 6.6.

A.5.6 Spherical and cylindrical PTFE bearings

(1) For maximum deformations of backing plates, see A.5.1.

(2) For eccentricities due to friction, rotation and lateral forces, see EN 1337-7, Annex A.

A.5.7 Details of installation

(1) Where structural components for load introduction from the bearings are not cast in situ directly on the bearing subsequent to its installation, e.g. in case of precast concrete or steel members, appropriate measures should be taken:

- to ensure their uniform contact with the bearing;
- to avoid areas of [AC1) variable rigidity for compression (AC1] on or underneath the bearing.

(2) Level corrections should be effected by grouting or suitable packing by plates with machined surfaces.

(3) More details are given in EN 1337-11.

Annex B [informative] – Technical specifications for expansion joints for road bridges

B.1 Scope

(1) This annex gives guidance on the preparation of technical specifications for expansion joints of road bridges.

NOTE: This annex is intended to be transferred to EN 1990 – Basis of structural design.

(2) The specification should include the following:

- movements (translational and rotational) from temperature, creep, shrinkage, traffic, and setting if relevant;
- traffic categories, other actions and environmental influences;
- the type of expansion joint and the related ETA;
- dimensions in sections and plan and categories of use (vehicles, cycles, pedestrians);
- particular requirements concerning durability, maintenance, accessibility and replacement, drainage, water tightness, noise emission.

(3) Data needed for the design of the connection between the expansion joint and the supporting structure of the bridge are those given in the relevant ETA and as supplied by the manufacturer for the specific project. They should include:

- dimensions including tolerances, movement capacities and other requirements for connections, the anchorage and method of installation;
- minimum requirements for stiffness of the main structure supporting the expansion joint;
- recommended detailing of the connection to the bridge;
- forces and moments from imposed movements to be taken into account in the bridge design.

NOTE 1: The following types of expansion joints are specified in the "Guideline for European Technical Approval of Expansion Joints for Road Bridges" (ETAG).

Table B.1: Types of expansion joints

ETAG-Part	Type
2	**Buried expansion joint:** This expansion joint is formed in situ using components such as waterproofing membranes or an elastomeric pad, to distribute the deformations to a greater width and to support the surfacing which is continuous over the deck joint gap. The components of the expansion joint are non flush with the running surface.
3	**Flexible expansion joint:** An in situ poured joint comprising a band of specially formulated flexible material (binder and aggregates), which also forms the surfacing, supported over the deck joint gap by thin metal plates or other suitable components. The joint material is flush with the running surface.
4	**Nosing expansion joint:** This expansion joint has lips or edges prepared with concrete, resin mortar or elastomeric compound. The gap between the edges is filled with a prefabricated flexible strip, which is non flush with the running surface.
5	**Mat expansion joint:** This expansion joint uses the elastic properties of a prefabricated elastomeric strip or pad to allow the expected movements of the structure. The strip is fixed by bolts to the structure. The joint element is flush with the running surface.
6	**Cantilever expansion joint:** This expansion joint consists of cantilever symmetrical and non-symmetrical elements (such as comb or saw tooth plates), which are anchored on one side of the deck joint gap and free at the other end bridge the deck joint gap. The elements are flush with the running surface.
7	**Supported expansion joint:** This expansion joint consists of one element flush with the running surface, which is fixed by hinges on one side and sliding supports on the other side (by a second element), and it spans the deck joint gap. The movement takes place through sliding on the non fixed side of the hinged element, i.e. on the supporting element, that is anchored to the substructure.
8	**Modular expansion joint:** This expansion joint consists of a number of watertight elements (in the traffic direction) comprising movement controlled metal beams supported by moveable substructures bridging the gap (i.e. crossbeams, cantilevers, pantographs). The metal beams are flush with the running surface.

NOTE 2: The ETAG on expansion joints for road bridges does not cover movable bridges.

NOTE 3: Expansion joints are normally installed either by the manufacturer or under the supervision of the manufacturer.

B.2 Technical specifications

B.2.1 General

(1) Expansion joints for bridges should be specified in accordance with the "Guideline for the Technical Approval of Expansion Joints for Road Bridges".

(2) The technical specifications for a specific bridge project should be based on the actions on the bridge and the bridge response to these actions.

NOTE: For the actions, combinations of actions and the modelling of the bridge structure to determine bridge responses relevant to expansion joints, see also Annex A.

(3) For drafting technical specifications the guideline for the preparation of the expansion joint schedule in B.2.2 should be used.

B.2.2 Expansion joint schedule

(1) An expansion joint schedule should give all the relevant information necessary for the design of the expansion joint, including the following:

1. Geometric data for the bridge deck surface and arrangement of the expansion joint in plan and section. The details should include provisions for the final alignment including durability of the load carrying connection between the expansion joint and the bridge structure. The schedule should also include provisions that allow to access to movable parts and that protect against corrosion and dirt.
2. User categories can be as follows:
   - vehicle;
   - cyclist;
   - pedestrian.

   Pedestrian paths may include maintenance vehicles, snow-ploughs, etc. Gaps and voids should be covered in such a way to prevent accidents from occurring.
3. Arrangement of the expansion joints in relation to the geometry of the bridge, e.g. longitudinal and transverse slope, curvature and the arrangement of bearings and the directions of their displacements.
4. Actions on expansion joints, [AC1) including persistent actions (AC1] and accidental actions comprising:
   - imposed displacements and rotations from the bridge movements in all directions corresponding to the installation temperature from the individual characteristic values of any transient, accidental and seismic actions on the bridge. For accidental and seismic actions the limit relating to opening or closing movements should be indicated;
   - imposed direct loads from user categories, vertical loads, horizontal loads for ultimate, serviceability and fatigue limit states;
   - environmental conditions that may affect the properties of the constituent materials.
5. Installation plan containing:
   - information about the prefixing (gap) of the expansion joint and its marking (considering the movement of the structure at the time of installation from creep, shrinkage, setting and the temperature assumed, e.g. + 10 °C);
   - requirements for adjustment measures to cope with the differences in assumptions (e.g. movements for $\Delta T = 1$ °C) in the form of a diagram;
   - temporary abutments and final abutments;
   - time of unfastening;
   - time of concreting.
6. Other requirements such as
   - assembly for erection, maintenance and repair;
   - provisions for anchorage and connections;
   - road restraint systems;
   - provisions for debris, dust, water;
   - accessibility to the joint and the drainage system;
   - design life according to traffic categories given in Table 4.5 of EN 1991-2;
   - jointing with the waterproofing system of the deck;
   - noise emission.

B.2.3 Actions for the design of the joint anchorage and connections

(1) The information needed from the joint manufacturer for the design of the anchorage or connections of the expansion joint is as follows:

1. Geometric data for the bearing surfaces of the expansion joint components including tolerances and types of connections for installation.
2. Minimum stiffness of the bearing surfaces.
3. Characteristic values of the forces and moments to be transmitted to the bridge structure.

B.3 Imposed loads, displacements and rotations from bridge movements

(1) The design values of displacements and rotations at the location of the expansion joints should be based on the rules specified in A.4.2 of Annex A.

(2) In calculating displacements and rotations the following aspects should be taken into account:

1. Relative displacements and rotations at both ends of the joint.
2. The angles between the longitudinal slope and the transverse slope of bridge surface and direction of movement of the movable bearings.
3. Effects of eccentricities.
4. Allowance for lifting the bridge to replace bearings (e.g. by 10 mm).

Annex C [informative] – Recommendations for the structural detailing of steel bridge decks

C.1 Highway bridges

C.1.1 General

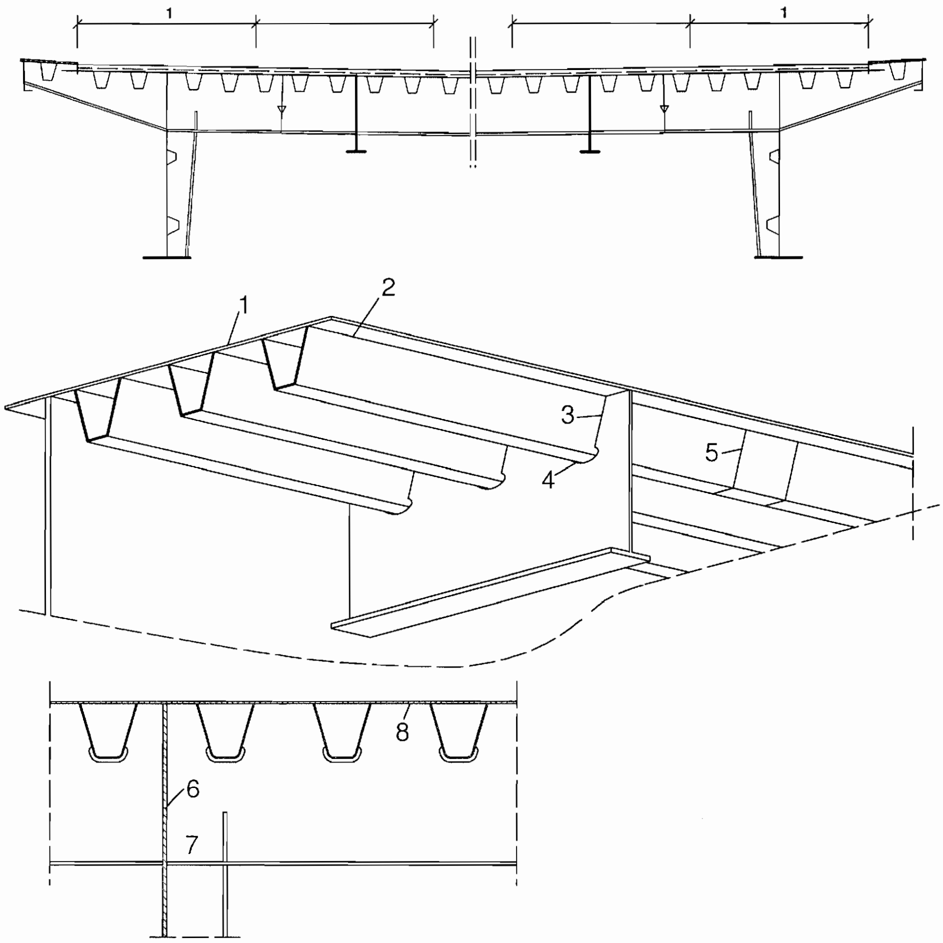
(1) This annex gives recommendations for the structural detailing and execution for road bridges to achieve a minimum quality standard as assumed in EN 1993-1-9.

NOTE 1: The rules related to execution are intended to be transferred to EN 1090.

NOTE 2: This annex applies to the types of details described in the following figures only.

(2) The recommendations are based on a standard design as given in Figure C.1 aiming at both durability of the steel structure and the surfacing. For the surfacing it is assumed that appropriate requirements for bonding, make up of the surfacing (material), plate preparation and waterproofing are met.

NOTE: AC1 The National Annex may give additional technical information. AC1



1 heavy traffic lane

1 deck plate

2 welded connection of stiffener to deck plate

3 welded connection of stiffener to web of crossbeam

4 cut out in web of crossbeam

5 splice of stiffener

6 splice of crossbeam

7 welded connection of crossbeam to main girder or transverse frame

8 welded connection of the web of crossbeam to the deck plate

Figure C.1: Examples of structural details in steel decks of highway bridges

NOTE: Annex C does not apply to decks provided with transverse stiffeners.

(3) The recommendations are based on the lanes in the carriageway carrying heavy traffic and cover:

1. the deck plate,
2. the welded connections of the stiffeners to the deck plate,
3. the welded connections of the stiffeners to the web of the crossbeam,
4. the detail of the cut out in the web of the crossbeam,
5. continuity of the stiffeners,
6. continuity of crossbeams,
7. the connection between crossbeams and main girders.

(4) Details of tolerances, testing methods and test requirements (including test results) are given in Tables C.3, Table C.4 and Table C.5.

C.1.2 Deck plate

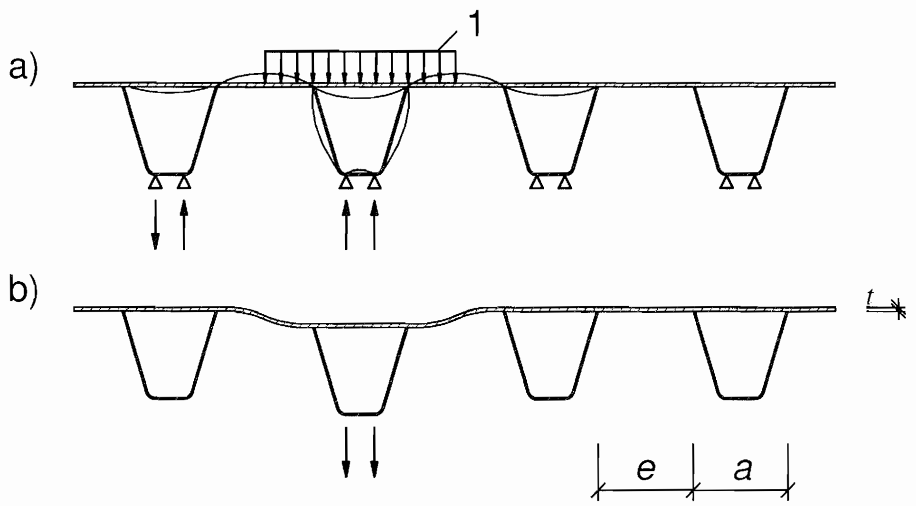
C.1.2.1 General

(1) Fatigue actions originate from bending of the deck plate due to wheel loads and tyre pressures, see Figure C.2.

(2) Figure C.2 a) illustrates the bending profile assuming the stiffeners will not deflect. Figure C.2 b) illustrates the effect of differential deflections of stiffeners.

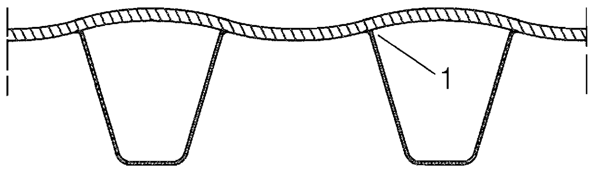
(3) The combination of the deck plate with the surfacing leads to an increase of the stiffness of the plate due to composite actions.

(4) Fatigue cracks may occur in the welds between the stiffeners and the plate, see Figure C.3, and in the surfacing.

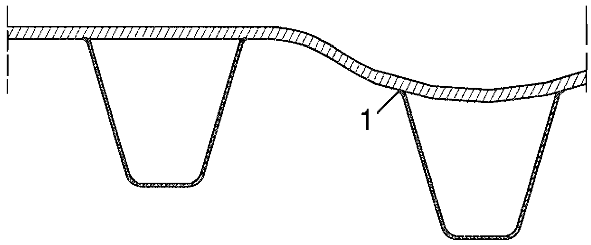


1 wheel load

Figure C.2: Effect of a) local wheel loads and b) differential deflections of stiffeners



a) crack initiation starting at weld root inside the stiffeners



b) crack initiation starting at weld toe outside the stiffeners

1 crack initiation

Figure C.3: Fatigue cracks in deck plate

(5) The recommendations relate to

1. the minimum thickness of the deck plate and the minimum stiffness of stiffeners
2. the splices of the deck plate
3. the connections between the deck plate and webs of main girders, webs of open section stiffeners and webs of crossbeams.

(6) The connection between the deck plate and the webs of the stiffeners is covered in section C.1.3.

(7) In order to achieve the tolerances for the assembly of the deck plate as specified in Table C.4, the tolerances given in Table C.3 (1) should be met.

C.1.2.2 Thickness of deck plates and minimum stiffness of stiffeners

(1) The thickness of deck plate should be selected according to the traffic category, the effects of composite action of the deck plate with the surfacing and the spacing of the supports of the deck plate.

NOTE 1: The National Annex may give guidance on the plate thickness to be used. Recommended plate dimensions are as follows, see Figure C.2:

1. Deck plate thickness in the carriage way in the heavy vehicle lane

   $t \geq 14$ mm for asphalt layer ≥ 70 mm,

   $t \geq 16$ mm for asphalt layer ≥ 40 mm.

2. Spacing of the supports of the deck plate by webs of stiffeners in the carriageway

   $e/t \leq 25$, recommended $e \leq 300$ mm.

   Locally e may be increased by 5 % where required, e.g. for adaptation to bridge curvature in plan.

3. Deck plate thickness for pedestrian bridges (with loads from maintenance vehicles):

   $t \geq 10$ mm and $e/t \leq 40$

   $e \leq 600$ mm.

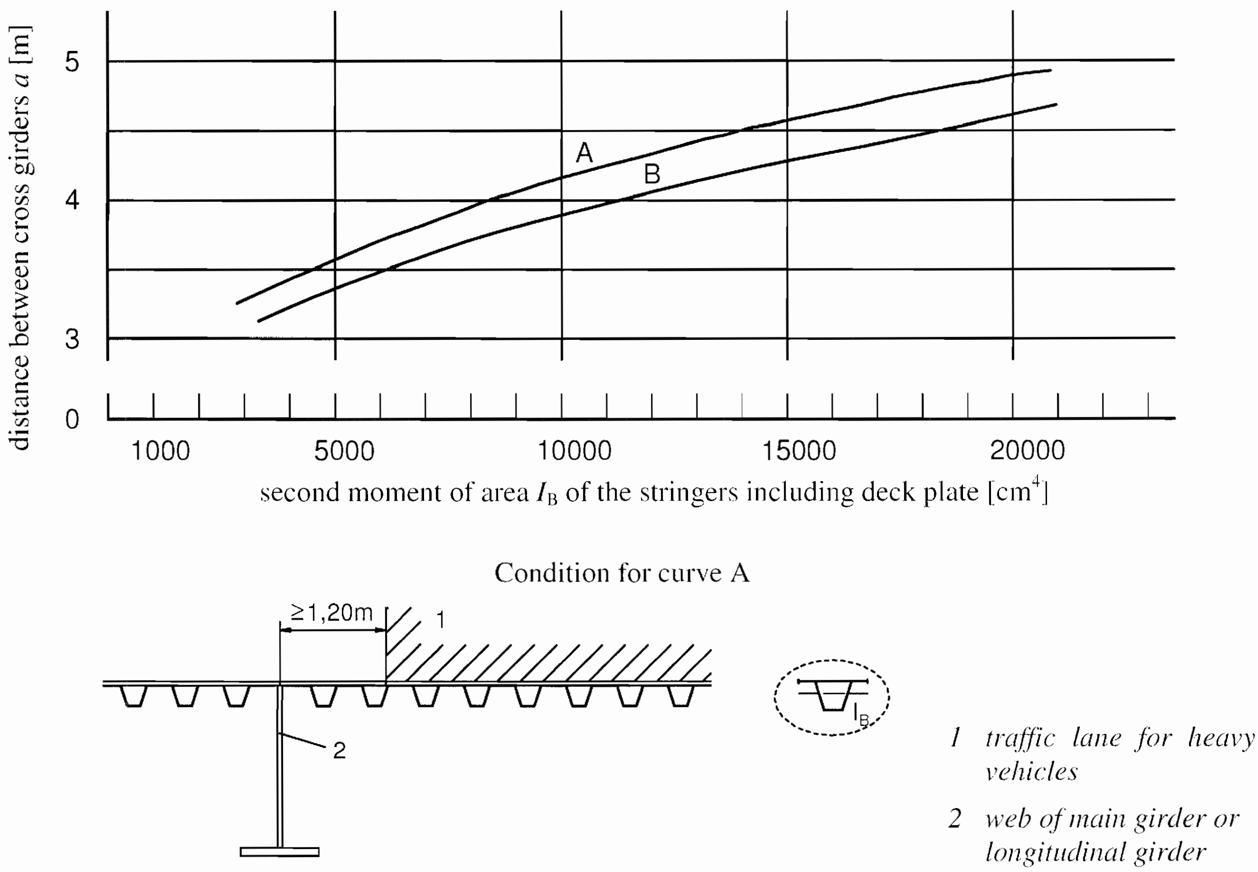
4. Thickness of stiffener:

$t_{\text{stiff}} \geq 6$ mm

NOTE 2: When the values given in NOTE 1 are satisfied, the bending moments in the deck plate do not need to be verified.

(2) The minimum stiffness of stiffeners should be selected in accordance with the traffic category and the location of the stiff bearing from webs of main girders or longitudinal girders in relation to the lane carrying heavy traffic. This is to prevent cracking of the surfacing due to differential deflections.

NOTE: The National Annex may give guidance on the minimum stiffness of stiffeners. The minimum stiffness values in Figure C.4 are recommended.



NOTE a) Curve A applies to all stiffeners, that are not covered by b).
b) Curve B applies to stiffeners that are located under the most heavily loaded traffic lane within 1,20 m of a web of a main girder
c) The figure applies to all types of stiffeners

Figure C.4: Minimum stiffness of longitudinal stiffeners

C.1.2.3 Deck plate welds

(1) Transverse welds (i.e. weld running across the traffic lane) should be double V-welds or single V-welds with root run or capping run or single V-welds with ceramic backing strips. Welds with metallic backing strips, see Figure C.6, are not recommended because of the discontinuity at stiffener locations.

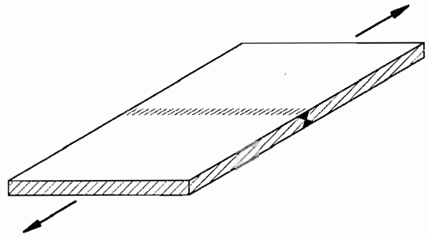


Figure C.5: Deck plate welds transverse to traffic lane without metallic backing strip

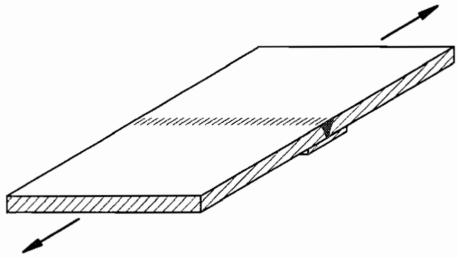
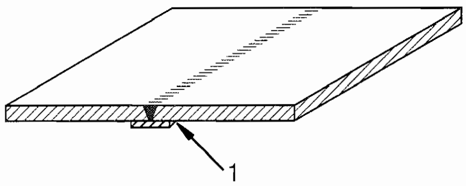


Figure C.6: Deck plate welds transverse to traffic lane with ceramic backing strip

(2) For tolerances and inspections of deck plate welds without backing strips see Table C.4 (1).

(3) Longitudinal welds (with welds running along the traffic lane) should be designed as transverse welds.



1 no sealing weld

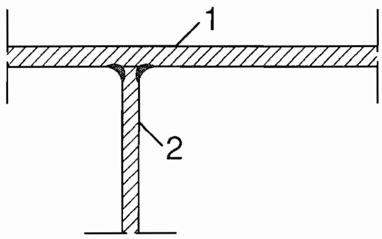
Figure C.7: Deck plate welds in the direction of traffic lane with metallic backing strip

(4) V-welds with metallic backing strips may be used for longitudinal welds with the following requirements:

1. execution in accordance with Figure C.7
2. tolerances and inspections given in Table C.4 (2).

C.1.2.4 Connection between the deck plate and webs of main girders, webs of open section stiffeners and webs of crossbeams

(1) The welds connecting the deck plate with the webs should be designed as fillet welds in accordance with Figure C.8.



1 deck plate

2 web of main girder

Figure C.8: Connection between the deck plate and the web of main girder

(2) For the connection of hollow section stiffeners to the deck plate, see C.1.3.

C.1.3 Stiffeners

C.1.3.1 Fatigue actions

(1) Fatigue actions result from:

1. bending in the webs imposed from the deformations of the deck plate due to rigid welded connections between the stiffener and the deck plate;
2. shear in the welds between stiffeners and deck plate from shear forces in the stiffeners;
3. longitudinal direct stresses in the stiffeners due to bending moments and axial forces in the stiffeners;
4. local bending at the connection between stiffeners and crossbeams in the web of the stiffeners and the webs of the crossbeams.

C.1.3.2 Type of stiffeners

(1) Stiffeners may either be closed section stiffeners, such as trapezoidal, V-shape, round or open stiffeners.

(2) For closed section stiffeners, see Table C.3 (2).

(3) For open stiffeners under traffic lanes, see Table C.3 (3).

(4) In the case of a change in the plate thickness of stiffeners, the misalignment at the surface of plates should not exceed 2 mm.

C.1.3.3 Stiffener to deck plate connection

(1) For closed section stiffeners under the carriageway the weld between the stiffener and the deck plate should be a butt weld.

(2) The throat thickness "a" should not be less than the thickness "t" of the stiffener, see Table C.4 (3) and (4).

(3) For stiffener to deck plate connections outside the carriageway, see Table C.4 (5).

(4) For tolerances and tests, see Table C.4 (3), (4) and (5).

C.1.3.4 Stiffener to stiffener connection

(1) The stiffener to stiffener connection should have splice plates in accordance with Table C.4 (6).

(2) The splice should be located close to the point of contraflexure of the stiffener (at a distance of 0,2 ℓ from crossbeam, where ℓ = span of stiffener).

(3) The welding sequence should be such that residual stresses are small and that the bottom flange of the stiffener receives residual compression. The welding sequence specified in Table C.4 (6) is as follows:

1. First weld between the stiffener and the splice plate.
2. Second weld between the stiffener and the splice plate; at [1] and [2] given in Table C.4 (6) at the bottom flange then the web should be welded.
3. Deck plate weld.

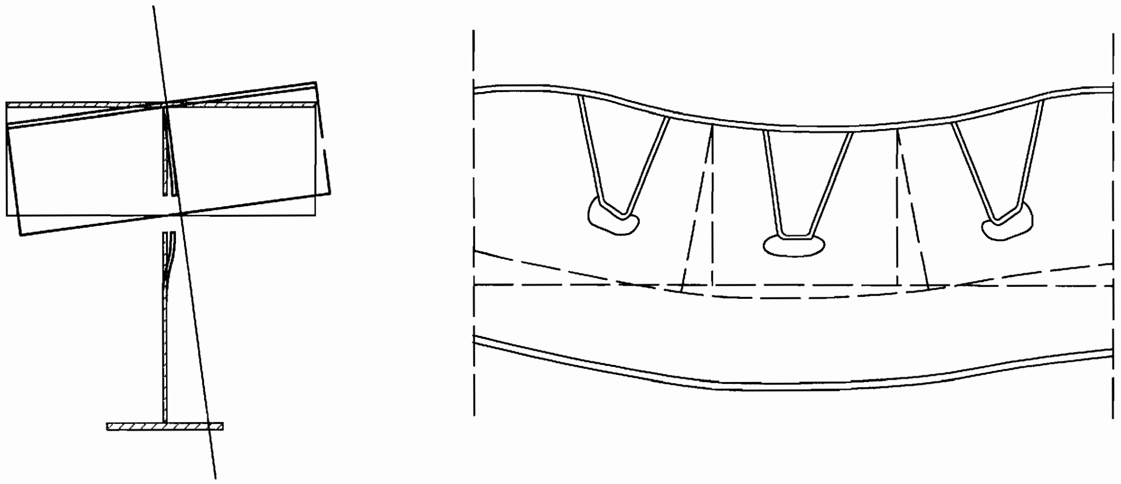
(4) For the butt welds between the stiffener and the splice plate the tolerances and inspections given in Table C.4 (7) should apply.

C.1.3.5 Connection of stiffeners to the web of the crossbeam

C.1.3.5.1 General

(1) Fatigue actions at the connection of the stiffeners to the web of the crossbeam result from the following, see Figure C.9:

1. Shear forces, torsional moments and stresses due to distortional deformations of the stiffeners induce stresses in the fillet welds between the stiffeners and the web of the crossbeam.
2. Rotations of the stiffeners due to deflections of the stiffeners induce bending stresses in the web. Poisson effects result in transverse deformations of the stiffeners restrained at the web of the crossbeam.
3. In plane stresses and strains in the web of the crossbeam may cause stress concentration at the edges of the cope holes and deformations on the stiffeners.



rotation of the stiffener at its connection to web of crossbeam, see C.1.3.5.1 (1) 2

imposed deformations to stiffener from strain distribution in the web of the crossbeam, see C.1.3.5.1(1) 3

Figure C.9: Connection of stiffeners to the web of the crossbeam

(2) The magnitude of these effects depends on whether the:

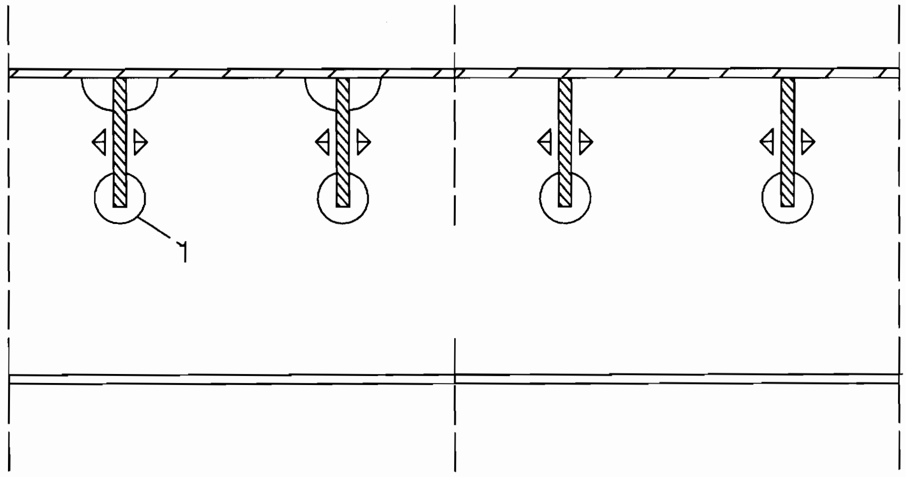
- stiffeners are passing through the web and the shapes of the cut out and cope hole;
- stiffeners are fitted between the webs of the crossbeams including the shape and fit up.

(3) Stiffeners should preferably pass through the webs of the crossbeam.

(4) Where it is not possible to pass the stiffeners through the webs, e.g. for bridges with extremely small depths of crossbeams or small spacing of crossbeams, stiffeners should be fitted between the webs in accordance with C.1.3.5.3.

(5) For flat stiffeners, see Figure C.10, the fatigue actions (see C.1.3.5.1 (1)) are similar to closed section stiffeners; however the effects of C.1.3.5.1 (1) 3. are smaller.

Open section stiffeners with longitudinal welds passing through the web of the crossbeam
with cope holes without cope holes



1 cope hole at bottom of flat to prevent melting of sharp edges

Figure C.10: Connections of flat stiffeners with the webs of crossbeams

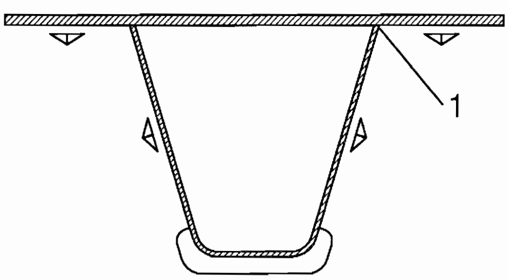
C.1.3.5.2 Cut outs in the webs of crossbeams

(1) For closed section stiffeners cut outs should be either with or without cope holes follows, see Figure C.11:

1. with cope holes around the soffit of the stiffener, see Figure C.11 a, with partial welding of the stiffener to the web;
2. without cope holes, see Figure C.11 b, with welding all around.

Figure C.11: Cut outs in the webs of crossbeams with or without cope holes

(2) Cope holes in the web of the crossbeam at the stiffener deck plate connections should be avoided, see Figure C.12.

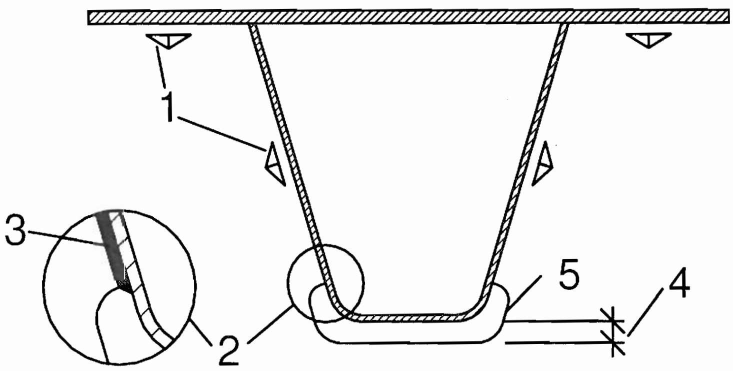


1 no cope holes
dimension according to Table C.4 (3), (4) and (5)

Figure C.12: Welded connections of the closed section stiffeners with web of crossbeam with cope holes

(3) The shape of the cut outs in the web of the crossbeam, see Figure C.13, should be such that:

1. The welds between the stiffeners and the web have adequate strength and the returns are without notches, see Figure C.13 a).
2. The dimensions of the cut out allow for:
   - stiffener profile tolerances, and
   - surface preparation, application and inspection of the corrosion protection, see Figure C.13 b).
3. The stress ranges $\Delta\sigma$ at the edge of the cut outs from in plane bending and out of plane bending of the web are within acceptable limits, see Figure C.13, c).



1 fillet welds

2 detail a)

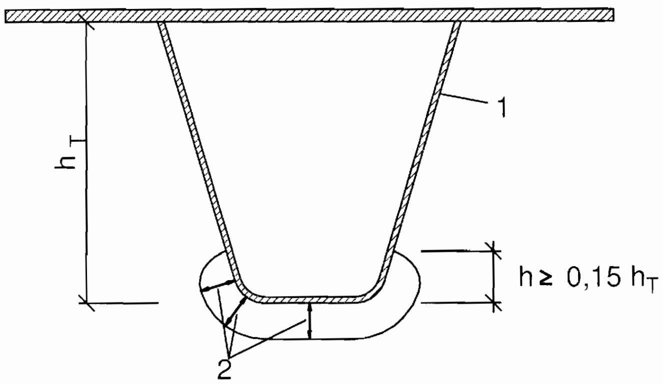
3 weld around the edge without notches, ground where necessary

4 detail b)

5 detail c)

Figure C.13: Critical details for the shape of cope holes

(4) The minimum size of the cut out should conform to ISO 12944-3 and Figure C.14.



1 plate thickness of web of crossbeam $t_{w,crossb}$

2 constant value of clearance $b \geq 2t_{w,crossb} \geq 25$ mm

Figure C.14: Minimum dimensions of cope holes

(5) The requirements for tolerance and inspection are given in Table C.4 (9).

(6) For the connection of the stiffeners to the end-crossbeam, see C.1.3.5.3.

(7) The requirements for the connection of stiffeners without cope holes are given in Table C.4 (8).

C.1.3.5.3 Stiffeners fitted between crossbeams

(1) Stiffeners may only be fitted between crossbeams, where the following conditions apply:

1. the bridge is designed for light traffic only, or the stiffeners are not located under the traffic;
2. the spacing between crossbeams is ≤ 2,75 m;
3. the grade of steel for the webs of the crossbeam conforms to the requirements for Z-quality given in EN 1993-1-10;
4. assembly and welding sequences are such that shrinkage effects are negligible.

(2) The connection of the stiffeners to the web should be made by butt welds with a weld preparation conforming to the requirements given in Table C.4 (10).

C.1.3.5.4 Stiffeners made of flat plates

(1) Flats passing through webs of crossbeams should have continuous fillet welds to the deck plate and should be welded to the web of the crossbeams on either sides, see [AC1) Figure C.10 (AC1].

(2) A maximum gap width of 1 mm is provided to reduce shrinkage.

(3) The requirements for detailing and inspection should be taken from Table C.4 (11).

C.1.4 Crossbeams

C.1.4.1 General

(1) Crossbeams should comprise the following:

1. web to stiffeners connection;
2. web to deck plate connection;
3. connection of the web of the crossbeam to the web of the main girder;
4. web to bottom flange connection of the crossbeam;
5. connection of the bottom flange of crossbeam to the web of main girder or to the bottom flange of main girder where both flanges are at an equal level;
6. connection of crossbeams to either transverse stiffeners, frames or diaphragms which are positioned in the same plane as the crossbeams.

(2) The radius of any corners of free edges of cut outs or cope holes should be measured.

(3) The detailed requirements given in C.1.4.2, C.1.4.3 and C.1.4.4 should be met.

C.1.4.2 Connections of the web of crossbeam

(1) The detailing and inspection of the welded connections of webs of crossbeams to the deck plate or to the web of the main girder should be carried out in accordance with Table C.4 (12) and Table C.4 (13), respectively.

(2) Splices of the webs in crossbeams should be welded in accordance with Table C.4 (14).

C.1.4.3 Connections of the flange of crossbeams

(1) The connection of the bottom flange of the crossbeam to the web of the main girder should be a butt weld conforming to Table C.4 (15).

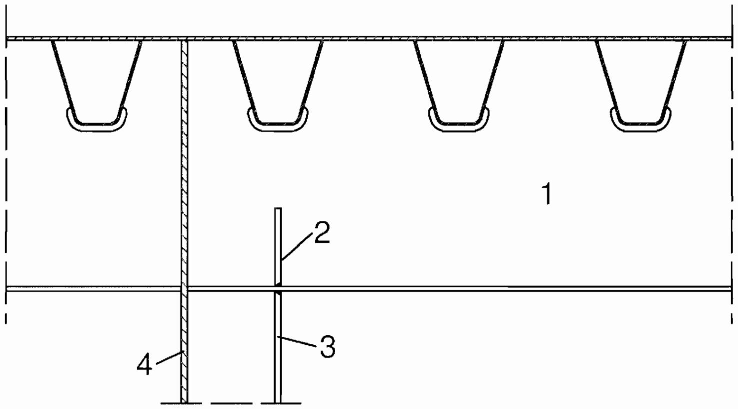
(2) Where the bottom flanges of the crossbeams and of the main girders are in the same plane, the connections should conform to Table C.4 (16).

(3) Flange to flange welded joints of crossbeams should conform to Table C.4 (14).

C.1.4.4 Transverse stiffeners, frames or diaphragms

(1) In order to reduce stress concentrations at connections between the crossbeams, transverse stiffeners and diaphragms, local stiffening should be provided at all connections and joints.

(2) Connections of components of transverse frames to crossbeams should be detailed in accordance with Figure C.15. The details should be verified for fatigue.



1 crossbeam

2 stiffener

3 transverse stiffener of web of main girder

4 web of main girder

Figure C.15: Typical connection of crossbeams to transverse stiffeners of web of main girders

C.2 Railway bridges

C.2.1 General

(1) C.2 gives recommendations for the design and structural detailing of orthotropic decks of railway bridges. It covers provisions for execution conforming to the quality standard as assumed in EN 1993-1-9.

(2) Bridge decks of railway bridges may consist of the following:

1. longitudinal stiffeners and crossbeams;
2. transverse stiffeners only.

(3) For bridge decks with longitudinal stiffeners, open section stiffeners made of flats or closed section stiffeners with trapezoidal profiles should be used.

(4) Crossbeams should be designed with bottom flanges for bridge decks with longitudinal closed section stiffeners. For bridge decks with longitudinal stiffeners made of flats, crossbeams may be designed without bottom flanges. For bridge decks with only transverse stiffeners, flat stiffeners may be used without bottom flanges.

C.2.2 Plate thickness and dimensions

(1) For bridge decks with longitudinal stiffeners and crossbeams, see Figure C.16, the dimensions in Table C.1 apply.

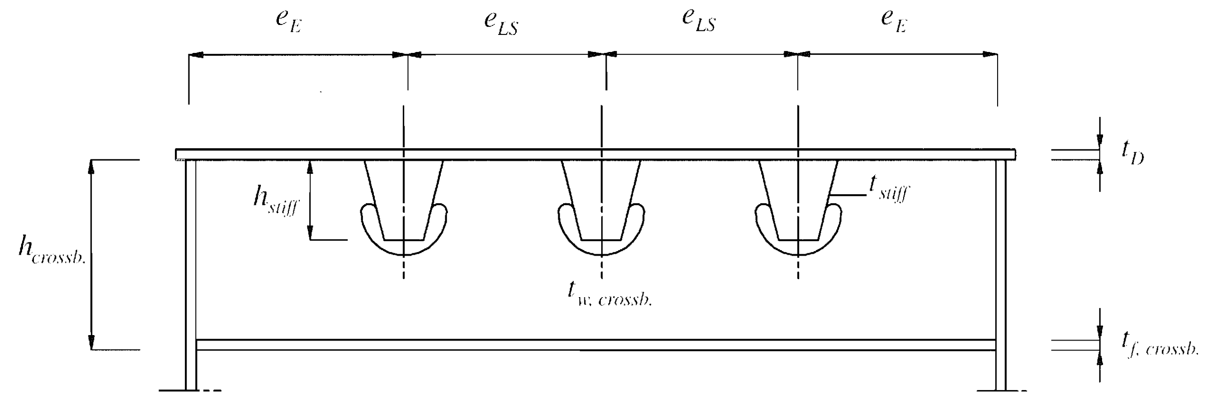


Figure C.16: Typical crossbeam details

Table C.1: Dimensions of bridge deck with longitudinal stiffeners

Dimensions	Open section stiffeners	Hollow section stiffeners
thickness of deck plate t_D	$t_D \geq 14$ mm	$t_D \geq 14$ mm
spacing e_{LS} between stiffeners	$e_{LS} \sim 400$ mm	600 mm $\leq e_{LS} \leq$ 900 mm
edge distance e_E of first stiffener	$e_E \geq e_{LS}$	$e_E \geq e_{LS}$
spacing of crossbeams e_{crossb}	$e_{crossb} \leq 2700$ mm	2500 mm $\leq e_{crossb} \leq$ 3500 mm
ratio of depth of stiffener to depth of crossbeam h_{stiff}/h_{crossb}	$h_{stiff}/h_{crossb} \leq 0{,}5$	$h_{stiff}/h_{crossb} \leq 0{,}4$
plate thickness t_{stiff}	$t_{stiff} \geq 10$ mm	6 mm $\leq t_{stiff} \leq$ 10 mm
plate thickness of web of crossbeam $t_{w.crossb}$	$t_{w.crossb} \geq 10$ mm	10 mm $\leq t_{w.crossb} \leq$ 20 mm
plate thickness of flange of crossbeam $t_{f.crossb}$	$t_{f.crossb} \geq 10$ mm	$t_{f.crossb} \geq 10$ mm

(2) For bridge decks with only transverse stiffeners, the dimensions in Table C.2 apply.

Table C.2: Dimensions of bridge deck with only transverse stiffeners only

thickness of deck plate t_D	$t_D \geq 14$ mm
spacing of crossbeams e_{crossb}	$e_{crossb} \sim 700$ mm
edge distance of crossbeams e_E	$e_E \geq 400$ mm
plate thickness of web crossbeam $t_{w.crossb}$	$t_{w.crossb} \geq 10$ mm
plate thickness of flange of crossbeam $t_{f.crossb}$	(where flanges are provided) $t_{f.crossb} \geq 10$ mm

C.2.3 Stiffener to crossbeam connection

(1) Longitudinal stiffeners should normally pass through the webs of crossbeams.

(2) The connections of open section stiffeners to the webs of crossbeams should be detailed as illustrated in Figure C.17.

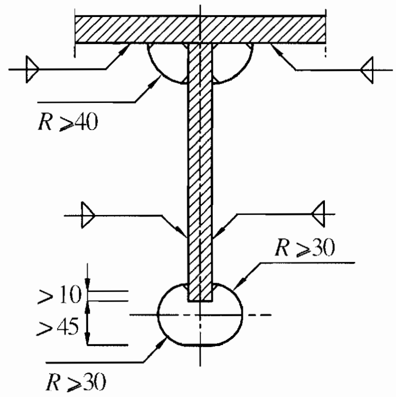
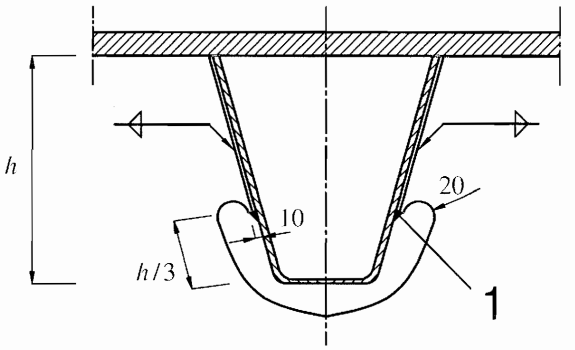


Figure C.17: Connection between the flat stiffener and the web of the crossbeam

(3) The connection of hollow section stiffeners to the webs of crossbeams should be detailed as illustrated in Figure C.18.



1 weld return, without notches, grinding where necessary

Figure C.18: Connection between the closed stiffener and the web of crossbeam

C.2.4 Weld preparation tolerances and inspections

C.2.4.1 General

(1) Unless specified otherwise Table C.3 and C.4 should be used for the structural detailing, weld preparation, tolerances and inspections of the bridge.

C.2.4.2 Stiffener to deck plate connections

C.2.4.2.1 Weld preparation of stiffeners

(1) For stiffener to deck plate connections, the edges of the formed plates (see Table C.4 (3) and (4)) should be chamfered, see Figure C.19.

(2) For plate of a thickness $t < 8$ mm, chamfering may be dispensed with provided that it can be verified (through welding tests) that the requirements for butt welds given in C.2.4.2.2 are met.

C.2.4.2.2 Requirements for butt welds

(1) The requirements for the butt welds should be as below:

- seam thickness a $\geq$ 0,9 t_{stiff}, see Table C.4(7);
- unwelded gap at root $\leq$ 0,25 t or $\leq$ 2 mm whichever is the smallest;

where a is the size of the weld;

t is the thickness of the plate;

t_{stiff} is the thickness of the stiffener.

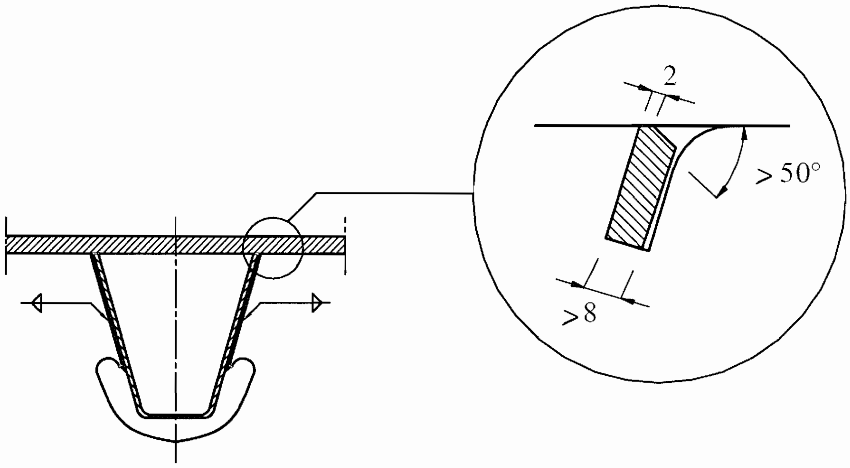


Figure C.19: Weld preparation of stiffener – deck plate connection

C.3 Tolerances for semi-finished products and fabrication

C.3.1 Tolerances for semi-finished products

(1) Irrespective of the fabrication methods employed for the delivery of the deck plate or formed stiffeners, the tolerances for fabrication specified in Table C.4 should be met.

(2) Table C.3 gives guidance on procurements. This is not necessary where the requirements of Table C.4 can be met by other measures.

C.3.2 Tolerances for fabrication

(1) The tolerances in Table C.4 apply for the design, fabrication and execution of bridge decks.

(2) In Table C.4, the following abbreviations are used:

- Requirement 1: External test results according to EN ISO 5817;
- Requirement 2: Internal test results according to EN ISO 5817;
- Requirement 3: See C.3.3;
- Requirement 4: Steels conforming to EN 10164 as specified in EN 1993-1-10.

C.3.3 Particular requirements for welded connections

(1) Where required in Table C.4, the conditions specified in Table C.5 apply in supplement to EN ISO 5817.

Table C.3: Tolerances of semi finished products

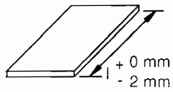
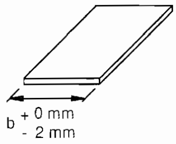
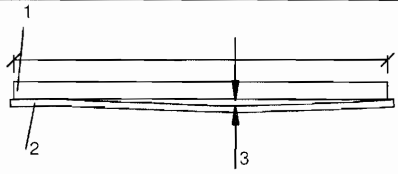
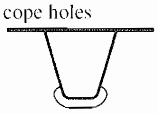
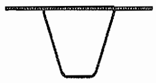
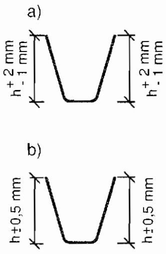
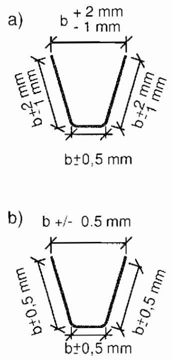
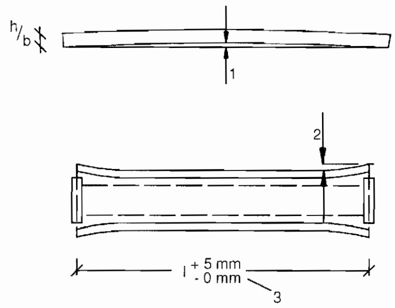
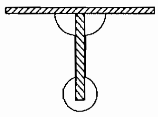
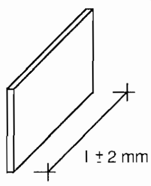
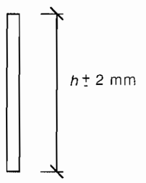
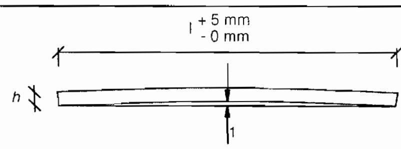
Product	Thickness	Length / depth	Width	Straightness	Remarks
1) Plate for deck after cutting and straightening by rolling	EN 10029, class C			 *1 measure length 2000 mm* *2 plate* *3 fit up gap max. 2,0 mm*	Length and widths inclusive of provisions for shrinkage and after application of the final weld preparation.
2) Formed profile a) for passing through crossbeams with cope holes  b) for passing through crossbeams without cope holes 	EN 10029, class C			a) and b)  *1 max. gap L/1000* *2 max widening + 1 mm* *3 for stiffener splices with splice plates* *radius r = r ± 2 mm* *rotation 1° on 4 m length* *parallelism 2 mm*	Plate thickness $t \geq 6$ mm For cold forming, only material suitable for cold forming is to be used. $R/t \geq 4$ for welding quality in cold forming region. The ends of the profiles are to be inspected visually for cracks and in case of any doubt by PT. ad b) If the tolerances are exceeded, the cut outs in the crossbeams are to be adapted to meet maximum gap width.
3) Flat profile for welding on both sides 	EN 10029, class C			 *1 max. gap L/1000*	Plate thickness $t \geq 10$ mm Choice of Z-quality conforming to EN 10164 from EN 1993-1-10 required.

Table C.4: Fabrication

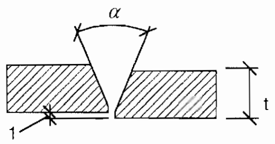
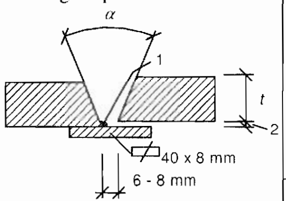
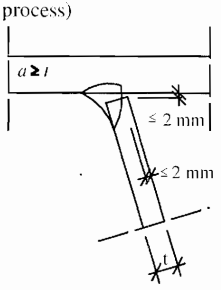
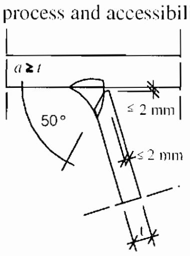
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
1) Splices of deck plate without backing strip  *1 misalignment ≤ 2 mm*	tensile stress $\sigma_{Ed} \leq 0{,}90\, f_{yk}$ and $\sigma_{Ed} > 0{,}75\, f_{yk}$	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 100 % ultrasonic (UT) or radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, maximum misalignment ≤ 2 mm ad 1b Requirement 1 and 3 ad 2 Requirement 2 and 3	Testing requirement, see C.3.3.
	tensile stress $\sigma_{Ed} \leq 0{,}75\, f_{yk}$ and $\sigma_{Ed} > 0{,}60\, f_{yk}$	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 100 % ultrasonic (UT) or radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, maximum misalignment ≤ 2 mm ad 1b Requirement 1 and 3 ad 2 Requirement 2 and 3	Testing requirement, see C.3.3.
	tensile stress $\sigma_{Ed} \leq 0{,}60\, f_{yk}$ or compression stress	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerances for weld preparation to be met, maximum misalignment ≤ 2 mm ad 1b Requirement 1 and 3	Testing requirement, see C.3.3.
2) Splices of deck plate with backing strip  *1 tack weld* *2 misalignment ≤ 2 mm* Weld preparation and weld preparation angle α in dependence of the welding process. Splices of metallic backing strips to be made of butt welds with grooved root and capping run. All work on splices to be finished before tack welding of deck plate. No sealing welds.	tensile stress $\sigma_{Ed} \leq 0{,}90\, f_{yk}$ and $\sigma_{Ed} > 0{,}75\, f_{yk}$	1a Inspection of weld preparation before welding; the melting of tack welds by subsequent weld beads to be verified by procedure tests 1b 100 % visual inspection after welding 2 100 % radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, tack welds of backing strips: Requirement 1 misalignment ≤ 2 mm ad 1b Requirement 1 fit up gaps between plate and backing strip ≤ 1 mm ad 2 Requirement 2 and 3	ad 1a Tack weld in the final butt weld, tack welds with cracks to be removed
	tensile stress $\sigma_{Ed} \leq 0{,}75\, f_{yk}$ and $\sigma_{Ed} > 0{,}60\, f_{yk}$	1a Inspection of weld preparation before welding 1b ≥ 50 % visual inspection after welding 2 10 % radiographic (RT) testing	ad 1a Tolerances for weld preparation to be met, tack welds of backing strips: Requirement 1 misalignment ≤ 2 mm ad 1b Requirement 1 and 3 ad 2 Requirement 2 and 3	ad 1a Tack weld in the final butt weld, tack welds with cracks to be removed
	tensile stress $\sigma_{Ed} \leq 0{,}60 f_{yk}$ or compression stress	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerances for weld preparation to be met, misalignment ≤ 2 mm ad 1b Requirement 1 and 3	
3) Stiffener-deckplate connection (fully mechanized welding process) 	independent on stress level in deck plate	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 Before fabrication: welding procedure tests conforming to EN ISO 15614-1 or when this is available, conforming to EN ISO 15613 with all welding heads. 3 During fabrication for each 120 m bridge 1 production test, however 1 production test for a bridge as a minimum, with all welding heads, checking by macro section tests	ad 1 Tolerances for weld preparations to be met ad 1b Requirement 1 ad 2 Fusion ratio to be met / Requirement 2 by preparing macro section tests (1 time at start or stop and one time at middle of weld) ad 3 see ad 2: however macro section tests only from middle of weld of the welding test	Starts and stops to be removed ad 2 Welding procedure tests under supervision of a recognized body, checking of welding parameters during fabrication ad 3 Execution, evaluation and documentation by fabricators production control, supervision by fabricators production control
4) Stiffener-deck plate connection (manual and partially mechanized welding process), weld preparation angle α in dependence of the welding process and accessibility 	independent on stress level in deck plate	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1 Tolerances for weld preparations to be met ad 1b Requirement 1	Starts and stops to be removed This requirement also applied to local welds, e.g. at stiffener-stiffener connections with splice plates, see 16).

Table C.4 (continued): Fabrication

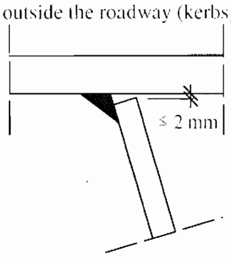
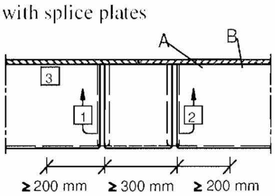
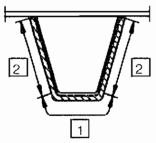
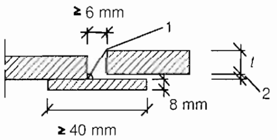
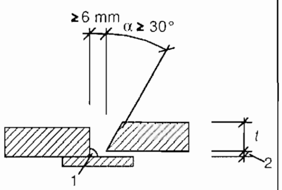
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
5) Stiffener-deck plate connection outside the roadway (kerbs)  throat thickness of fillet weld a as required by analysis	pedestrian loading without loading by vehicles except errant vehicles	1a Inspection of weld preparation before welding 1b ≥ 25 % visual inspection after welding 2 Measuring of throat thickness	ad 1a Tolerance of gap to be met ad 1b Requirement 1 ad 2 Requirement of throat thickness to be met and requirement 1	Starts and stops to be removed
6) Stiffener-stiffener connection with splice plates   *A site weld* *B shop weld*	independent on stress level	1a Inspection of weld preparation before welding 1b = 100 % visual inspection after welding	ad 1a Tolerance of gap to be met, misalignment between stiffener and splice plate ≤ 2 mm ad 1b Requirement 1 and 3	The non welded length of the seam on site between stiffeners and deck plate may also be provided at one side of the splice only. ad 1a For the root gaps see detail 7), for the site weld see details 3), 4) and 5)
7) Stiffener to stiffener connection with splice plates a) for plate thicknesses t = 6 - 8 mm  *1 continuous tack weld* *2 misalignment ≤ 2 mm* b) for plate thicknesses t ≥ 8 mm  *1 continuous tack weld* *2 misalignment ≤ 2 mm* weld preparation angle α dependant on welding process and gap width dependant on plate thickness	independent on stress level	1a Inspection of weld preparation before welding 1b = 100 % visual inspection after welding 2 Test of weld by 1 production test	ad 1a Tolerance of weld preparation to be met, misalignment ≤ 2 mm ad 1b Requirement 1 ad 2 Requirement 1 and 2	

Table C.4 (continued): Fabrication

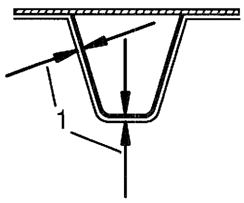
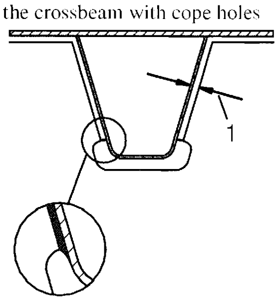
Structural detail	Stress level σ_{Ed}, τ_{Ed}	Testing method and amount of testing	Test results required	Remarks
8) Stiffener-crossbeam connection with stiffeners passing through the crossbeam without cope holes  *1 gap ≤ 3 mm*	throat thickness $a = a_{nom}$ according to analysis for gap width $s \leq 2$ mm, for greater gap widths s: $a = a_{nom} + (s-2)$ minimum throat thickness $a = 4$ mm	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, required throat thickness a available ad 1b Requirement 1 and 3	1. It is assumed, that first the stiffeners are welded to the deck plate (with jigs and fixtures) and the crossbeams are subsequently assembled and welded. 2. The tolerances for the cut outs of crossbeams follow the tolerances of the formed profiles for the stiffeners, see Table C.3, detail 2)b). 3. The cut edges of the webs of crossbeams should be without notches, in case there are they should be ground. For flame cutting EN ISO 9013 – Quality 1 applies.
9) Stiffener-crossbeam connection with stiffeners passing through the crossbeam with cope holes  *1 gap ≤ 3 mm* welds around edges of cope holes without notches	throat thickness $a = a_{nom}$ according to analysis for gap width ≤ 2 mm, for greater gap widths s: $a = a_{nom} + (s-2)$ minimum throat thickness $a = 4$ mm	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, required throat thickness a available ad 1b Requirement 1 and 3	1. It is assumed, that first the stiffeners are welded to the deckplate (with jigs and fixtures) and the crossbeams are subsequently assembled and welded. 2. The tolerances for the cut outs of crossbeams follow the tolerances of the formed profiles for the stiffeners, see Table C.3, detail 2)a). 3. The cut edges of the webs of crossbeams including the cope holes should be without notches, in case there are they should be ground. For flame cutting EN ISO 9013 – Quality 1 applies.

Table C.4 (continued): Fabrication

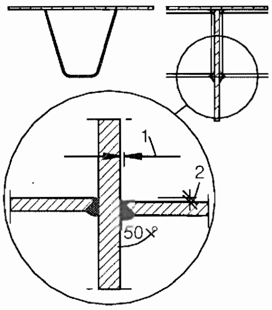
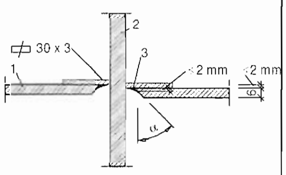
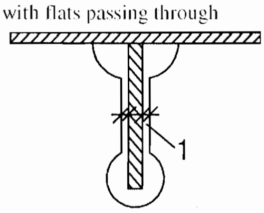
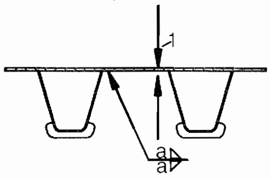
Structural detail	Stress level σ_{Ed}, τ_{Ed}	Testing method and amount of testing	Test results required	Remarks
10) Stiffener-crossbeam connection with stiffeners fitted between crossbeams (not passing through)  *1 gap ≤ 2 mm* *2 misalignment ≤ 2 mm* single sided full penetration weld (single V-weld) without backing strip  *1 stiffener* *2 web of crossbeam* *3 tack weld* single sided full penetration weld with backing strip	throat thickness $a > t_{stiffener}$	1a Inspection of weld preparation before welding 1b ≥ 50 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, misalignment ≤ 2 mm ad 1b Requirement 1 and 3	1. This solution is only permitted for bridges with light traffic and for crossbeam spacing ≤ 2,75 m. 2. Webs of crossbeams see requirement 4. 3. The sequence of assembly and welding of stiffeners and crossbeams should be decided to prevent harmful shrinkage effects. 4. Backing strips in one part, see 7). 5. Tack welds only inside final welds.
11) Stiffener-crossbeam connection with flats passing through  *1 gap ≤ 1 mm*	throat thickness of fillet welds according to analysis	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met ad 1b Requirement 1 and 2	The cut edges of the crossbeam should be prepared without notches and hardening, else they should be ground. For flame cutting EN ISO 9013 – quality 1 applies.
12) Connection of web of crossbeam to deck plate (with or without cope holes)  *1 gap ≤ 1 mm*	throat thickness of fillet welds according to analysis	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, requirement 1 and 2 ad 1b Requirement 1	The flame cut edges should be prepared in accordance with EN ISO 9013 – quality 1.

Table C.4 (continued): Fabrication

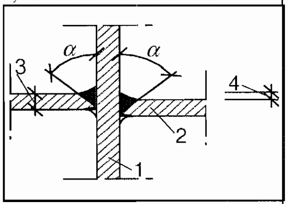
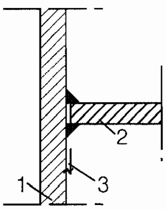
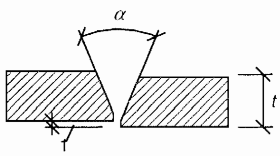
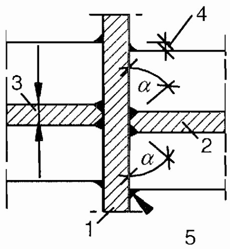
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
13) Connection of webs of crossbeams to web of main girder a) for continuous crossbeams  *1 web of main girder* *2 web of crossbeam* *3 $t_{w,crossb}$* *4 misalignment $\leq 0,5\ t_{w,crossb}$*	independent on stress level	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, requirement 1 for a), misalignment $\leq 0.5\ t_{web,cross\ beam}$ ad 1b Requirement 1	Execution with full penetration welds, weld preparation angle α and weld preparation in accordance with welding process and plate thickness.
b) for non continuous crossbeams  *1 web of main girder* *2 web of crossbeam* *3 gap ≤ 2 mm*	throat thickness of fillet weld according to analysis	see above	ad 1a see above ad 1b see above	Execution with fillet welds, see detail 12)
14) Splice of lower flange or web of crossbeam  *1 misalignment 0 – 2 mm*	independent on stress level	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding 2 ≥ 10 % ultrasonic (UT) or radiographic (RT) testing	ad 1a Tolerance of weld preparation to be met, requirement 1, misalignment ≤ 2 mm ad 1b Requirement 1 and 3 2 Requirement 2	
15) Connection of crossbeam flanges to web of main girder  *1 web of main girder* *2 web of crossbeam* *3 $t_{w,crossb}$* *4 misalignment $\leq 0,5\ t_{w,cross}$* *5* $r \geq \frac{t_{w,maingirder}}{2} \geq 8$	independent on stress level	1a Inspection of weld preparation before welding 1b 100 % visual inspection after welding	ad 1a Tolerance of weld preparation to be met, misalignment $0,5 \leq t_{web,cross\ beam}$ ad 1b Requirement 1 and 3	1. Webs of main girders, requirement 4. 2. For smaller plate thicknesses also single V-welds with root run and capping run may be used, see 13). 3. Only full penetration butt welds with root run and capping run should be used.

Table C.4 (continued): Fabrication

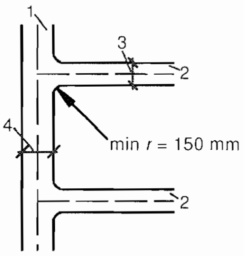
Structural detail	Stress level σ_{Ed}	Testing method and amount of testing	Test results required	Remarks
16) In plane connection of flanges of crossbeams and main girders  *1 main girder* *2 crossbeam* *3* b_{crossb} *4* $b_{main\ girder}$	minimum radius at connection min r = 150 mm, all plate thicknesses are equal otherwise a fatigue assessment is necessary			Transitions to be ground.

Table C.5: Conditions supplementary to EN ISO 5817

To No.	Discontinuity	Supplementary requirement
3	Porosity and gas pores	only singular small pores acceptable
4	Localized (clustered) porosity	maximum sum of pores: 2 %
5	Gas canal, long pores	no larger long pores
10	Bad fit up, fillet welds	transverse welds to be tested totally, small root reset only locally acceptable
		$b \leq 0{,}3 + 0{,}10$ a, however $b \leq 1$ mm b = root gap or root reset respectively
11	Undercut	a) butt welds only locally acceptable $h \leq 0{,}5$ mm b) fillet welds not acceptable where transverse to stress direction, undercuts have to be removed by grinding.
18	Linear misalignment of edges	maximum 2 mm sharp edges to be removed
24	Stray flash or arc strike	not acceptable outside fusion zone
26	Multiple discontinuities in a cross section	not allowed
6	Solid inclusions	not allowed
25	Welding spatter	spatter and their heat affected zones to be removed

Annex D [informative] – Buckling lengths of members in bridges and assumptions for geometrical imperfections

D.1 General

(1) This annex gives buckling length factors β that may be used for the design of compression members in bridges in the expression:

$$\ell_K = \beta \; L \tag{D.1}$$

(2) This Annex also gives guidance for the application of imperfections for second order analysis, see 5.3.2 of EN 1993-1-1.

(3) Imperfections may either be determined from the relevant buckling mode, [AC1) see 5.3.2(11) of EN 1993-1-1 (AC1] or from simplified assumptions for member imperfections, see 5.3.2(3) of EN 1993-1-1.

D.2 Trusses

D.2.1 Vertical and diagonal elements with fixed ends

(1) Unless more accurately verified, the following values with regard to the relative stiffness and the nature of connections may be used:

- for in plane buckling: $\beta = 0{,}9$
- for out of plane buckling: $\beta = 1{,}0$

D.2.2 Vertical elements being part of a frame, see Figure D.1a) or D.1b)

(1) The buckling length factor β may be taken from Table D.1

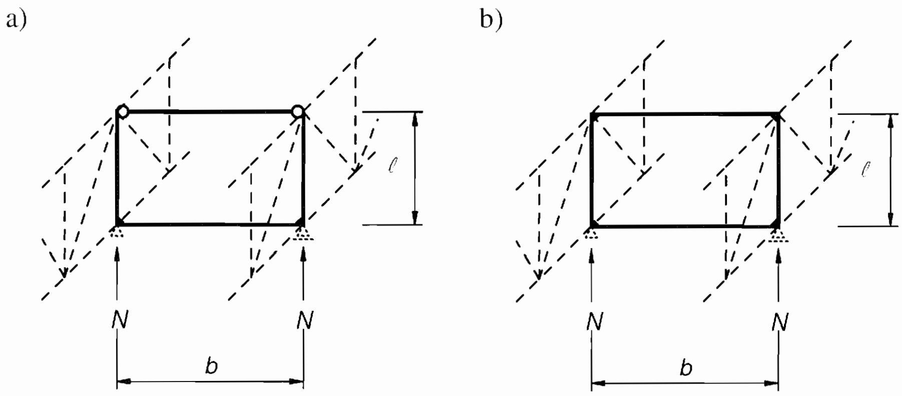
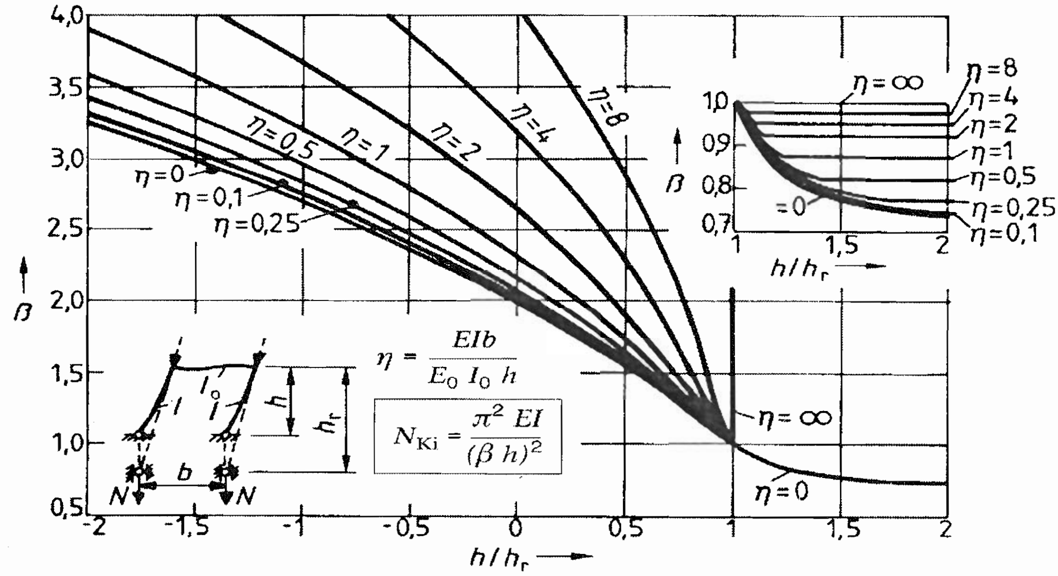
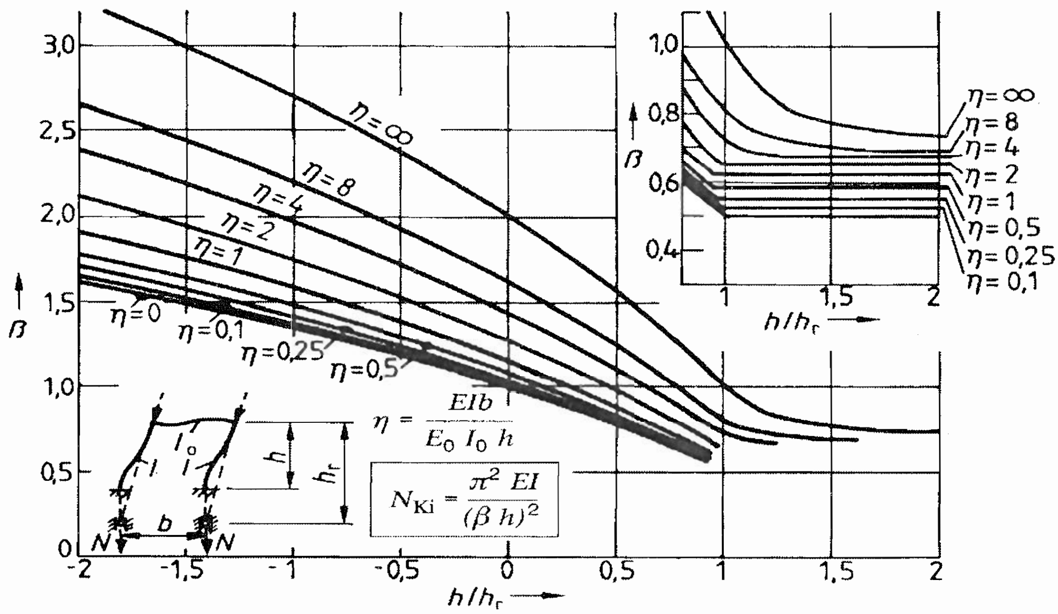
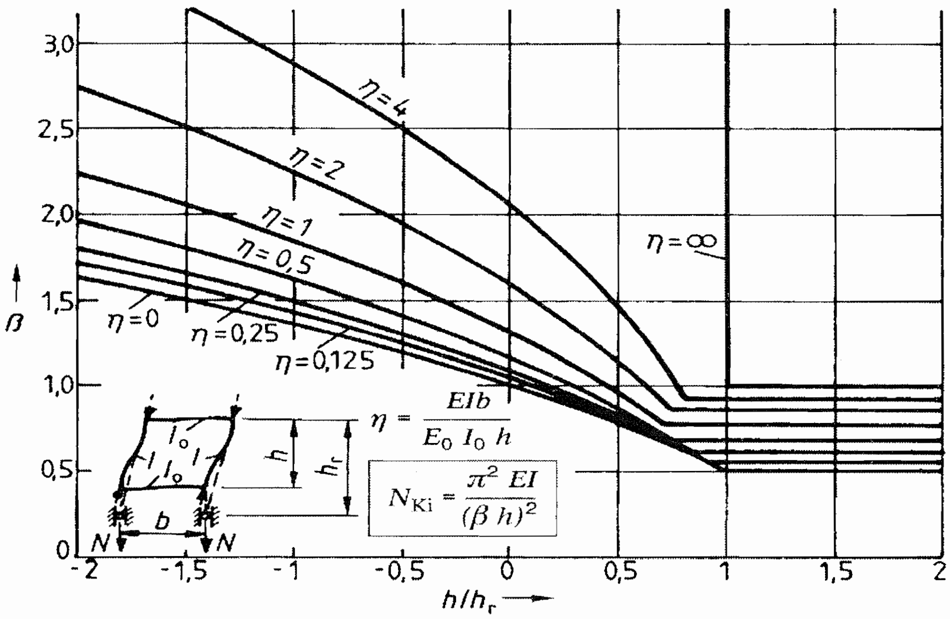


Figure D.1: Vertical elements being part of a frame

Table D.1: Buckling length factors β





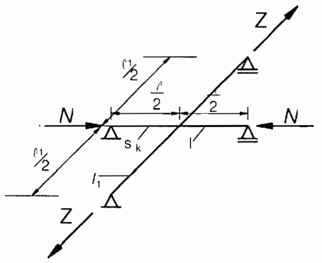
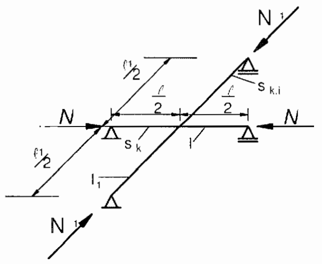
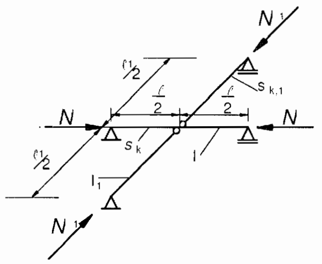
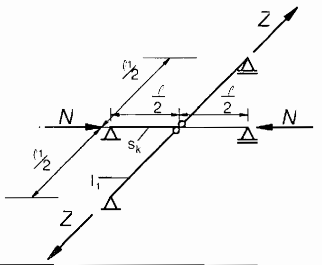
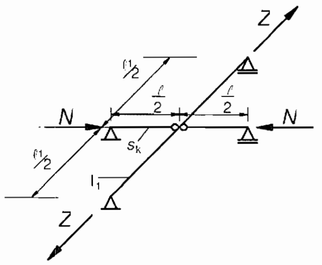
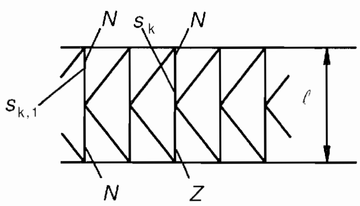


D.2.3 Out of plane buckling of diagonals

(1) The buckling lengths of diagonals of trusses may be taken from Table D.2.

(2) Connections should be effective in both stiffness and strength in bending to achieve continuity of diagonals, as given in Table D.2.

Table D.2: Buckling lengths

	1	2	3
1		$\beta=\sqrt{\dfrac{1-\dfrac{3}{4}\dfrac{Z\ell}{N\ell_1}}{1+\dfrac{I_1\ell^3}{I\ell_1^3}}}$ but $\beta \geq 0{,}5$	
2		$\beta=\sqrt{\dfrac{1+\dfrac{N_1\ell}{N\ell_1}}{1+\dfrac{I_1\ell^3}{I\ell_1^3}}}$ but $\beta \geq 0{,}5$	$\beta_1=\sqrt{\dfrac{1+\dfrac{N\ell_1}{N_1\ell}}{1+\dfrac{I\ell_1^3}{I_1\ell^3}}}$ but $\beta_1 \geq 0{,}5$
3		continuous compression members $\beta=\sqrt{1+\dfrac{\pi^2}{12}\dfrac{N_1\ell}{N\ell_1}}$	hinged compression members $\beta_1 = 0{,}5$ when $EI \geq \dfrac{N_1\ell^3}{\pi^2\ell_1}\left(\dfrac{\pi^2}{12}+\dfrac{N\ell_1}{N_1\ell}\right)$
4		$\beta=\sqrt{1-0{,}75\dfrac{Z\ell}{N\ell_1}}$ but $\beta \geq 0{,}5$	
5		$\beta = 0{,}5$ when $\dfrac{N\ell_1}{Z\ell} \leq 1$ or when $EI_1 \geq \dfrac{3Z\ell_1^2}{4\pi^2}\left(\dfrac{N\ell_1}{Z\ell}-1\right)$	
6		$\beta=\left(0{,}75-0{,}25\left\lvert\dfrac{Z}{N}\right\rvert\right)$ but $\beta \geq 0{,}5$	$\beta_1=\left(0{,}75-0{,}25\dfrac{N_1}{N}\right)$ $N_1 < N$

(3) For diagonals which are elastically supported at midspan, see Figure D.2 and equation (D.2):

$$\beta = \sqrt{1 - \frac{3}{16}\frac{C\,L}{N}} \tag{D.2}$$

where L is the system length;

N is the maximum of N_1 or N_2;

C is the lateral support stiffness but $C \leq \dfrac{4N}{\ell}$

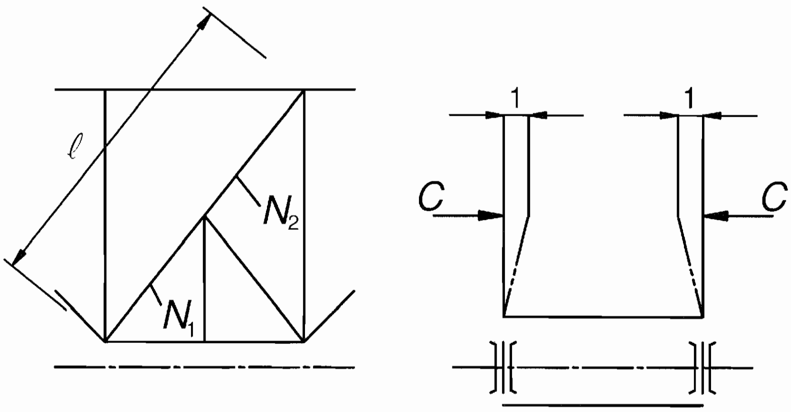


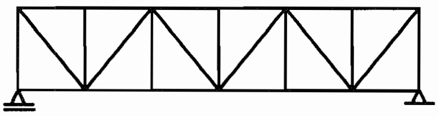
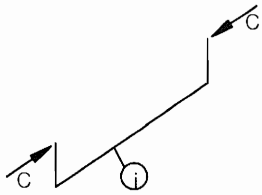
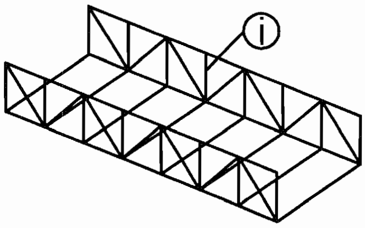
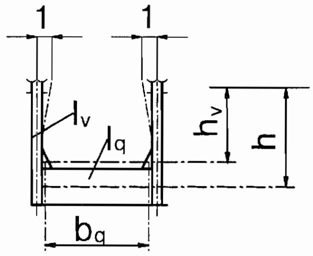
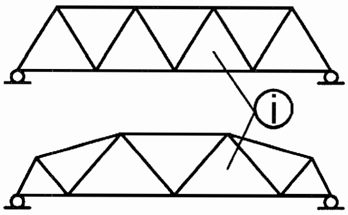
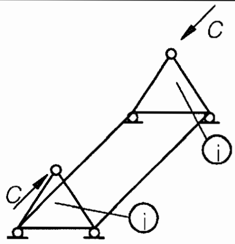
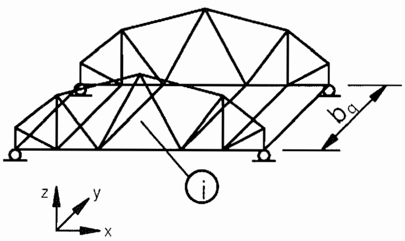
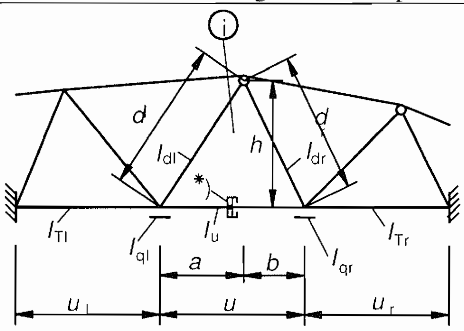
Figure D.2: Diagonal with elastical support at midspan

D.2.4 Compression chords of open bridges

(1) Compression chords may be modelled as columns with lateral supports.

(2) The stiffness of the lateral supports may be determined using Table D.3.

Table D.3: Lateral stiffnesses C_d [AC1) for trusses (AC1]

	1	2
1	 Example of truss bridges with posts	
1a	 Modelling	 $C = \dfrac{EI_v}{\dfrac{h_v^3}{3} + \dfrac{h^2 b_q I_v}{2I_q}}$
2	 Example of truss bridges without posts	 2U-frame in truss bridges without posts
2a	 Modelling	 *) torsional hinge Modelling: bottom chord of the U-frame with [AC1) flexural stiffness EI_ℓ only, adjacent bottom chords with torsional stiffness GI_T (AC1]
2b	spring stiffness $C = \dfrac{A+B-2D}{AB-D^2} EI_u$	
2c	$A = \dfrac{h^2 I_u}{n_\ell} + \dfrac{d_\ell^3 I_u}{3I_{d\ell}} + \dfrac{a^2 u}{3}$ $\quad n_\ell = \dfrac{2}{b_q} I_{q\ell} + \dfrac{GI_{T\ell}}{Eu_\ell}$ $B = \dfrac{h^2 I_u}{n_r} + \dfrac{d_r^3 I_u}{3I_{dr}} + \dfrac{b^2 u}{3}$ $\quad n_r = \dfrac{2}{b_q} I_{qr} + \dfrac{GI_{Tr}}{Eu_r}$ $D = \dfrac{1}{6} abu$ The length d_ℓ, d_r, a, b, u and b_q may be reduced in case of rigid ends. u_ℓ and u_r may be reduced in case of end that are torsionally rigid. $EI_{d\ell}$, EI_{dr}, EI_u = bending stiffness of diagonals and bottom chords for out-of-plane bending $EI_{q\ell}$, EI_{qr} = bending stiffness of the crossbeam $GI_{T\ell}$, GI_{Tr} = St. Venant torsional stiffness of the adjacent chords	

D.3 Arched Bridges

D.3.1 General

(1) In D.3.1, buckling length factors β are given for in plane and out of plane buckling of arches.

(2) The critical buckling force N_{cr} in the arch for in plane buckling is expressed by:

$$N_{cr} = \left(\frac{\pi}{\beta s}\right)^2 EI_y \tag{D.3}$$

where N_{cr} relates to the force at the supports;
s is the half length of the arch;
EI_y is the in plane flexural stiffness of the arch;
β is the buckling length factor.

(3) The critical buckling force in free standing arches for out of plane buckling is expressed by:

$$N_{cr} = \left(\frac{\pi}{\beta \cdot \ell}\right)^2 EI_z \tag{D.4}$$

where N_{cr} relates to the force at the supports:
ℓ is the projection length of the arch:
EI_z is the out of plane flexural stiffness of the arch:
β is the buckling length factor.

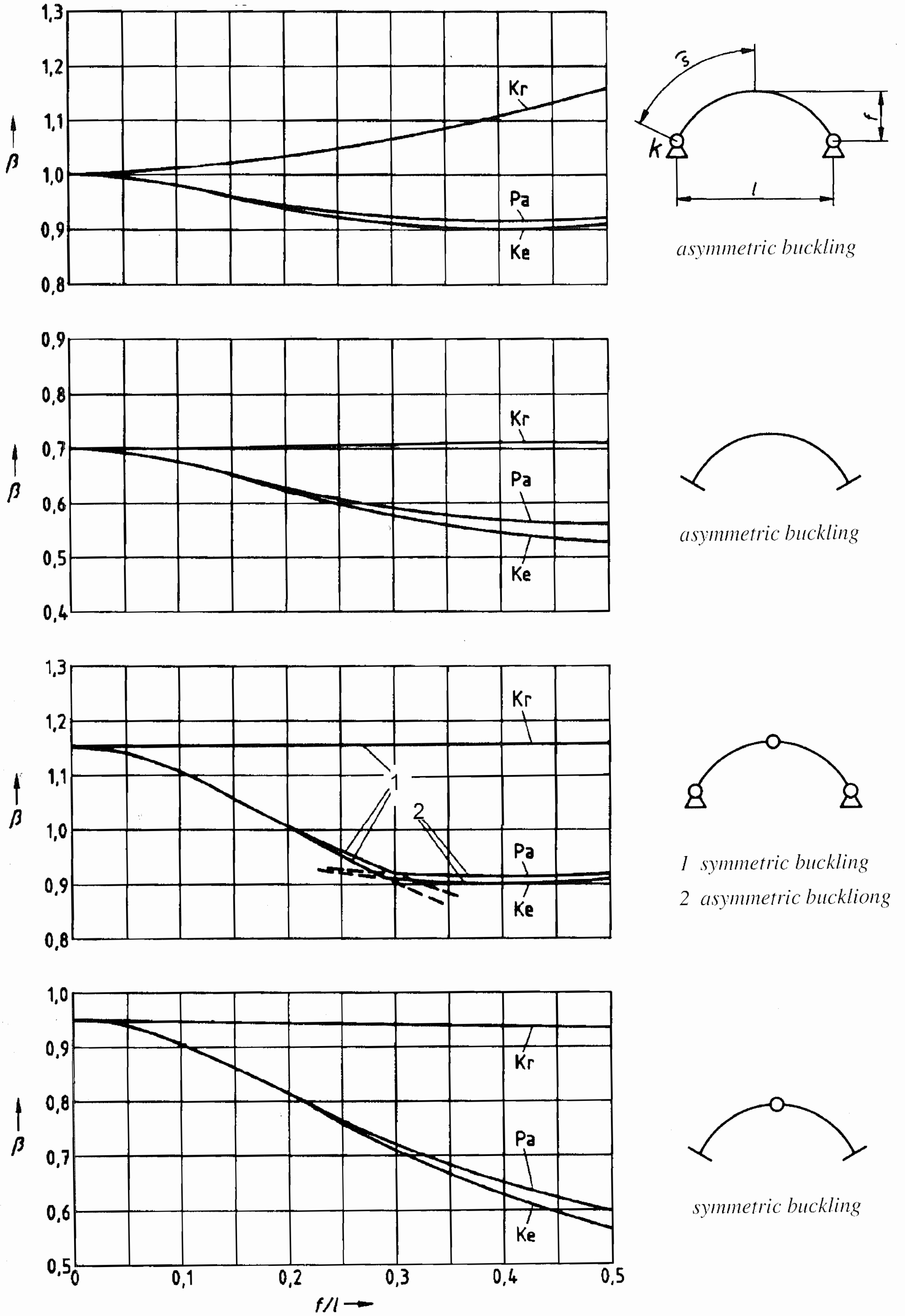
(4) The out of plane buckling of arches with wind bracing and portals may be verified by a stability check of the end portals.

D.3.2 In plane buckling factors for arches

(1) For arches [AC1) with hinged or rigid supports (AC1] buckling factors β are given in Table D.4.

(2) For arches with a tension tie and hangers buckling factors β are given in Figure D.4.

Table D.4: Buckling length factor β for arches for $f/\ell > 0,1$



Pa: parabolic form Ke: chain form Kr: circular form
for Pa and Ke the loading is vertical
AC1 for Kr the radial loading (e.g. hydrostatic pressure) is supposed AC1

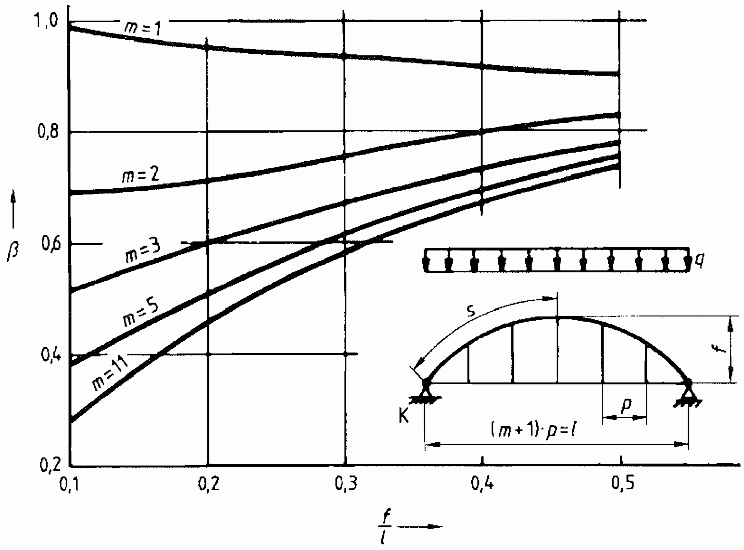


Figure D.4: Buckling factor β

(3) Snap through of arches may be assumed to be prevented, if the following criterion is satisfied:

$$\ell \sqrt{\frac{EA}{12EI_y}} > K \qquad \text{(D.5)}$$

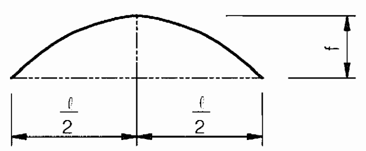
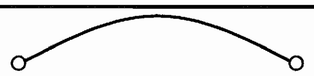
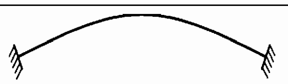
where A is the cross sectional area

I_y is the moment of inertia

K is a factor

(4) The factor K may be taken from Table D.5.

Table D.5: Factor K

	f/ℓ	0,05	0,075	0,10	0,15	0,20
	K	35	23	17	10	8
		319	97	42	13	6

D.3.3 Out of plane buckling factors for free standing arches

(1) For out of plane buckling of free standing arches the buckling factors may be taken as:

$$\beta = \beta_1 \beta_2 \qquad \text{(D.6)}$$

where β_1 is given in Table D.6 and β_2 is given in Table D.7

Table D.6: β_1 - values

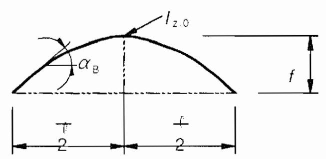
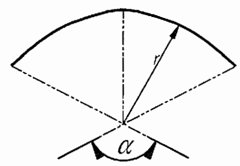
f/ℓ	0,05	0,10	0,20	0,30	0,40	
I_z constant	0,50	0,54	0,65	0,82	1,07	
I_z varies $I_z(\alpha_B)=\frac{I_{z,0}}{\cos\alpha_B}$	0,50	0,52	0,59	0,71	0,86	

Table D.7: β_2 - values

Loading	β_2	Comments
conservative (The deck is fixed to the top of the arch)	1	
by hangers	$1-0{,}35\frac{q_H}{q}$	q total load
by posts	AC1 $1+0{,}45\frac{q_{St}}{q}$ AC1	q_H load part transmitted by hangers q_{St} load part transmitted by posts

(2) For out of plane buckling of free standing circular arches with radial loading the buckling factor β may be taken as

$$\beta = \pi r \alpha \frac{\sqrt{\pi^2+\alpha^2 K}}{\ell(\pi^2-\alpha^2)} \tag{D.7}$$

where r is the radius of the circle

α is the section angle of the arch $0 < \alpha < \pi$

$K = \frac{EI_z}{GI_T}$

D.3.4 Out of plane buckling of arches with wind bracing and end portals

(1) The out of plane buckling may be verified by a stability check of the end portals carried out in accordance with D.2.2.

(2) The buckling length factor β may be taken from Table D.1, using the geometry in Figure D.5.

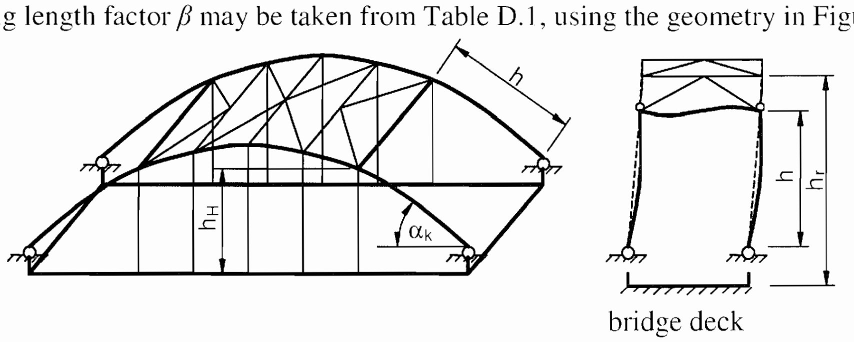


Figure D.5: Buckling of portals for arches

(3) The value h_r in Table D.1 may be taken as AC1 the mean of all lengths h_H of the hangers multiplied by $\frac{1}{\sin\alpha_k}$, where α_k (constant) is taken from Figure D.5. AC1

D.3.5 Imperfections

(1) Unless the relevant buckling modes are used for imperfection, see [AC1⟩5.3.2(11) of EN 1993-1-1,⟨AC1] the bow imperfections given in Table D.8 for in plane buckling of arches and in Table D.9 for out of plane buckling of arches may be used.

Table D.8: Shape and amplitudes of imperfections for in plane buckling of arches

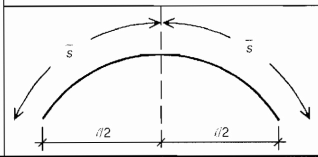
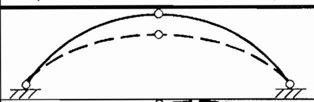
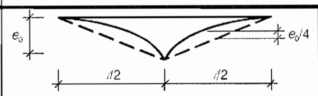
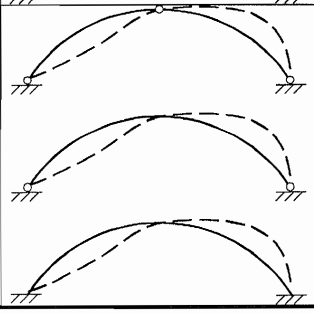
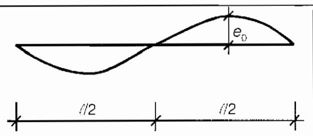
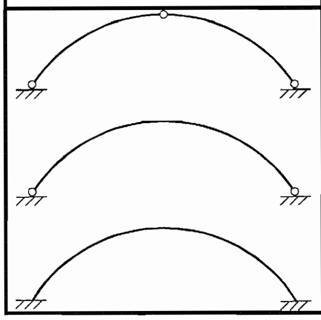
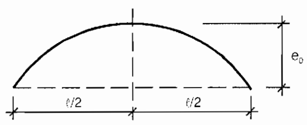
	1	2	3			
		shape of imperfection (sinus or parabola)	e_0 according to classification of cross section to buckling curve			
			a	b	c	d
1			$\frac{s}{300}$	$\frac{s}{250}$	$\frac{s}{200}$	$\frac{s}{150}$
2			$\frac{\ell}{600}$	$\frac{\ell}{500}$	$\frac{\ell}{400}$	$\frac{\ell}{300}$

Table D.9: Shape and amplitudes of imperfections for out of plane buckling of arches

	shape of imperfection (sinus or parabola)	e_0 according to classification of cross section to buckling curve				
			a	b	c	d
		$\ell \le 20$ m	$\frac{\ell}{300}$	$\frac{\ell}{250}$	$\frac{\ell}{200}$	$\frac{\ell}{150}$
		$\ell > 20$ m $\ell_1 = \sqrt{20\,\ell\,[\text{m}]}$	$\frac{\ell_1}{300}$	$\frac{\ell_1}{250}$	$\frac{\ell_1}{200}$	$\frac{\ell_1}{150}$

Annex E [informative] – Combination of effects from local wheel and tyre loads and from global traffic loads on road bridges

E.1 Combination rule for global and local load effects

(1) When considering the local strength of stiffeners of orthotropic decks, effects from local wheel and tyre loads acting on the stiffener and from global traffic loads acting on the bridge should be taken into account (see Figure E.1).

(2) To take into account the different sources of these loads the following combination rule may be applied to determine the design values:

$$\sigma_{Ed} = \sigma_{loc,d} + \psi \sigma_{glob,d} \tag{E.1}$$

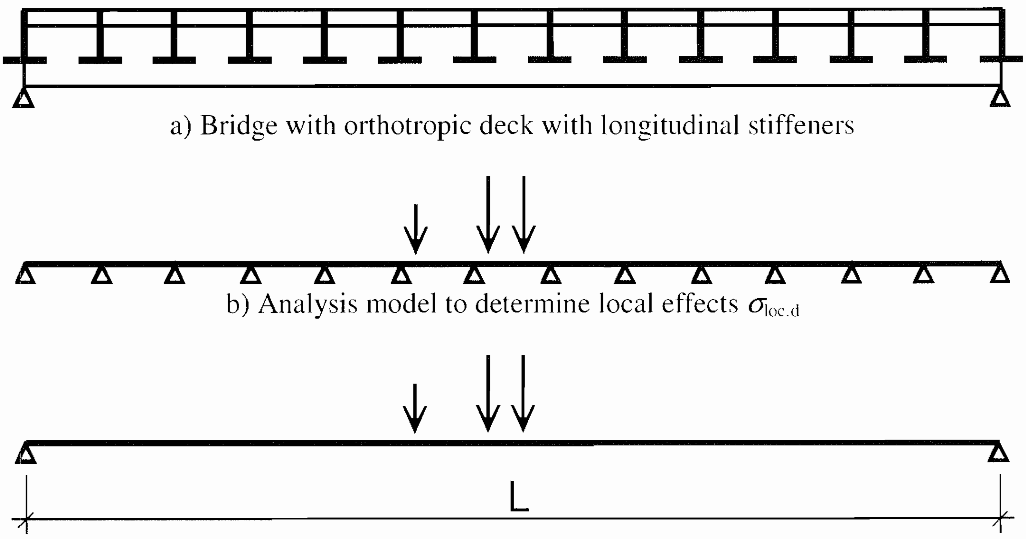
$$\sigma_{Ed} = \psi \sigma_{loc,d} + \sigma_{glob,d} \tag{E.2}$$

where σ_{Ed} is the design value of stress in the stringer due to combined effects of local load [AC1⟩ $\sigma_{loc.Ed}$ and global load $\sigma_{glob.Ed}$ ⟨AC1];

$\sigma_{loc.d}$ is the design value of stress in the stringer due to local wheel or tyre load from a single heavy vehicle;

$\sigma_{glob.d}$ is the design value of stress in the stringer due to bridge loads comprising one or more heavy vehicles;

ψ is the combination factor.



a) Bridge with orthotropic deck with longitudinal stiffeners

b) Analysis model to determine local effects $\sigma_{loc.d}$

c) Analysis model to determine global effects $\sigma_{glob.d}$

Figure E.1: Modelling of structure with local and global effects

E.2 Combination factor

(1) The combination factor ψ may be determined on the basis of the weight distributions of several lorries acting on an influence line for combined action effects.

NOTE: The National Annex may give guidance on the combination factor. The factor in Figure E.2 is recommended.

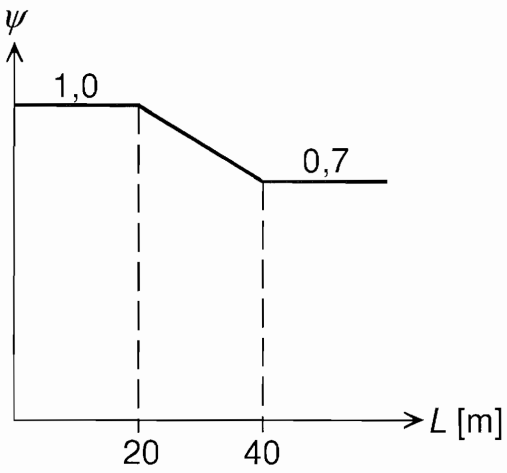


Figure E.2: Combination factor dependent on span length *L*

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