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Eurocode 3 — Design of steel structures — Part 2: Bridges

Eurocode 3 — Bemessung und Konstruktion von Stahlbauten — Teil 2: Brücken

Eurocode 3 — Calcul des structures en acier — Partie 2: Ponts

ICS:

CCMC will prepare and attach the official title page.

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European foreword

This document (prEN 1993‑2:2024) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1993‑2:2006.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognise the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

Introduction

## 0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of Parts:

* EN 1990, *Eurocode — Basis of structural and geotechnical 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*
* EN 1998, *Eurocode 8 — Design of structures for earthquake resistance*
* EN 1999, *Eurocode 9 — Design of aluminium structures*
* New parts are under development, e.g. Eurocode for design of structural glass

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

## 0.2 Introduction to EN 1993 (all parts)

EN 1993 applies to the design of buildings and civil engineering works in steel. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 — Basis of structural and geotechnical design.

EN 1993 is concerned only with requirements for resistance, serviceability, durability and fire resistance of steel structures. Other requirements, e.g. concerning thermal or sound insulation, are not covered.

EN 1993 is subdivided in various parts:

EN 1993‑1, *Design of Steel Structures — Part 1: General rules and rules for buildings;*

EN 1993‑2, *Design of Steel Structures — Part 2: Bridges;*

EN 1993‑3, *Design of Steel Structures — Part 3: Towers, masts and chimneys;*

EN 1993‑4, *Design of Steel Structures — Part 4: Silos and tanks;*

EN 1993‑5, *Design of Steel Structures — Part 5: Piling;*

EN 1993‑6, *Design of Steel Structures — Part 6: Crane supporting structures;*

EN 1993‑7, *Design of steel structures — Part 7: Sandwich panels.*

(4) EN 1993‑1 in itself does not exist as a physical document, but comprises the following 14 separate parts, the basic part being EN 1993‑1‑1:

EN 1993‑1‑1, *Design of Steel Structures — Part 1‑1: General rules and rules for buildings;*

EN 1993‑1‑2, *Design of Steel Structures — Part 1‑2: Structural fire design;*

EN 1993‑1‑3, *Design of Steel Structures — Part 1‑3: Cold-formed members and sheeting;*

NOTE Cold formed hollow sections supplied according to EN 10219 are covered in EN 1993‑1‑1.

EN 1993‑1‑4, *Design of Steel Structures — Part 1‑4: Stainless steel structures;*

EN 1993-1-5, *Design of Steel Structures — Part 1‑5: Plated structural elements;*

EN 1993‑1‑6, *Design of Steel Structures — Part 1‑6: Strength and stability of shell structures;*

EN 1993‑1‑7, *Design of Steel Structures — Part 1‑7: Plate assemblies with elements under transverse loads;*

EN 1993‑1‑8, *Design of Steel Structures — Part 1‑8: Joints;*

EN 1993‑1‑9, *Design of Steel Structures — Part 1‑9: Fatigue;*

EN 1993‑1‑10, *Design of Steel Structures — Part 1‑10: Material toughness and through-thickness properties;*

EN 1993‑1‑11, *Design of Steel Structures — Part 1‑11: Tension components;*

EN 1993‑1‑12, *Design of Steel Structures — Part 1‑12: Additional rules for steel grades up to S960;*

EN 1993‑1‑13, *Design of Steel Structures — Part 1‑13: Beams with large web openings;*

EN 1993‑1‑14, *Design of Steel Structures — Part 1‑14: Design assisted by finite element analysis.*

All subsequent parts EN 1993‑1‑2 to EN 1993‑1‑14 treat general topics that are independent from the structural type like structural fire design, cold-formed members and sheeting, stainless steels, plated structural elements, etc.

All subsequent parts numbered EN 1993‑2 to EN 1993‑7 treat topics relevant for a specific structural type like steel bridges, towers, masts and chimneys, silos and tanks, piling, crane supporting structures, etc. EN 1993‑2 to EN 1993‑7 refer to the generic rules in EN 1993‑1 and supplement them.

## 0.3 Introduction to EN 1993‑2

EN 1993‑2 is the second part 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.

EN 1993‑2 is intended to be used with Eurocodes EN 1990, EN 1991 (all parts) 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.

## 0.4 Verbal forms used in the Eurocodes

The verb “shall” expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regu-lation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may” expresses a course of action permissible within the limits of the Eurocodes.

The verb “can” expresses possibility and capability; it is used for statements of fact and clarification of concepts.

## 0.5 National annex for prEN 1993‑2

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing prEN 1993‑2 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this standard, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

National choice is allowed in prEN 1993‑2 through notes to the following clauses:

|  |  |  |  |
| --- | --- | --- | --- |
| 4.1.3(2) | 4.3(2) – 2 choices | 4.4(1) | 4.4(2) |
| 5.2.1(1) – 2 choices | 5.2.3(2) | 5.2.4(1) | 5.3.1.2(1) |
| 5.3.2(2) | 5.4(1) | 5.5(1) | 5.6 (1) |
| 5.7(1) | 5.7(3) | 6(3) | 6(7) |
| 7.4.1(2) | 8.2.2.3(1) | 8.2.2.5(1) | 8.3.5(6) |
| 9.1(3) | 9.3(1) | 9.4(1) | 10.1.1(1) |
| 10.1.1(3) | 10.1.2(2) | 10.1.2(3) | 10.1.3(3) |
| 10.2.2(1) | 10.2.3(1) | 10.4.2(1) | 10.4.3 (2) – 2 choices |
| 10.4.3(3) | 10.5(1) – 2 choices | 10.6(1) | 11.1.2(1) |
| 11.1.3(1) | 11.1.5(1) | 11.2.2(1) | 11.2.3(1) |
| 11.2.4(1) | 11.2.6(1) | 11.3(1) | 11.4(1) |
| C.1(1) | C.3.1(2) | C.3.2.2(1) | C.3.2.2(2) |
| E.4(1) | F.1(2) |  |  |

National choice is allowed in EN 1993‑2 on the application of the following informative annexes:

|  |  |  |  |
| --- | --- | --- | --- |
| Annex C | Annex F |  |  |

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

# Scope

## Scope of EN 1993‑2

(1) EN 1993‑2 provides rules for structural design of steel bridges and steel parts of steel-concrete composite bridges.

(2) EN 1993‑2 is applicable to the resistance, serviceability and durability of steel bridge structures.

(3) The design of tension components and related parts is covered by EN 1993‑1‑11. For the design of hangers for tied-arch bridges, the additional provisions in Annex A apply.

(4) Supplementary requirements for seismic design are given in EN 1998‑2.

## Assumptions

(1) Unless specifically stated, EN 1990, EN 1991 (all parts), EN 1998 (all parts) and EN 1993‑1 (all parts) apply.

(2) The design methods given in EN 1993‑2 are applicable if:

* the execution quality is as specified in EN 1090‑2 and EN 1090‑4, and
* the construction materials and products used are as specified in the relevant parts of EN 1993, or in the relevant material and product specifications.

# Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in ‘should’ clauses), permissions (i.e. through ‘may’ clauses), possibilities (‘can’ clauses), and in notes.

EN 1090‑2, *Execution of steel structures and aluminium structures — Part 2: Technical requirements for steel structures*

EN 1090‑4, *Execution of steel structures and aluminium structures — Part 4: Technical requirements for cold-formed structural steel elements and cold-formed structures for roof, ceiling, floor and wall applications*

EN 1990:2023[[1]](#footnote-1)*, Eurocode — Basis of structural and geotechnical design*

EN 1991 (all parts), *Eurocode 1 — Actions on structures*

EN 1993 (all parts)*, Eurocode 3 — Design of steel structures*

EN 1998 (all parts), *Eurocode 8 —* *Design of structures for earthquake resistance*

# Terms, definitions and symbols

## Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, EN 1993‑1 (all parts) and the following apply.

3.1.1

breathing (of plates)

out-of-plane deformation of a plate caused by repeated application of in-plane loading

3.1.2

secondary structural elements

structural elements that do not form part of the main load-carrying structures of the bridge, such as guard rails, parapets, maintenance walkways, ladders and access covers

## Symbols

### General

For the purpose of this document, the symbols given in EN 1990, EN 1993‑1 (all parts) and the following apply.

### Latin upper-case symbols

|  |  |
| --- | --- |
| *A* | cross-sectional area |
| *A*eff | effective area |
| *A*wc | area of the compression zone of the web |
| *A*1c*, A*2c | cross-section areas of a crossbeam submitted to local load from stiffeners |
| *A*k, *B*k | parameters for the calculation of the critical shear buckling factor |
| *A*χ, *B*χ, *C*χ | parameters for the calculation of the reduction factor for shear buckling |
| *C*U | height of cope hole above base of stiffener |
| *C*d | spring stiffness |
| *C*L | height of cope hole below base of stiffener |
| *C*zf | coefficient for the design buckling resistance of curved plate girders |
| *D* | hanger diameter |
| *D*0 | reference value of the hanger diameter |
| *E* | modulus of elasticity |
| *F*Ed | additional lateral force |
| *F*b,Rd | design bearing resistance for ultimate limit states verifications |
| *F*v,Ed,ser | shear force in a bolt under the characteristic load combination at serviceability limit state |
| *F*i*, F*i+1 | loads introduced by the stiffeners at section i and section i+1 |
| *F*1*, F*2 | point loads |
| *I* | moment of inertia (second moment of area) |
| *I*B | Moment of inertia (second moment of area) of the stiffeners including deck plate |
| *L* | span length |
| *L*cr | buckling length |
| *L*E | embedded length |
| *L*F | clear height of gusset plate |
| *L*i | span length of the span i |
| *L*R | spacing of the centres of rigid lateral restraints to the compression flange |
| *L*w | half wavelength of buckling |
| *L*w,i | effective correlation length |
| *L*1 | modified span length of an arch |
| *L*λ | critical length of the influence line or area |
| *M*cr | elastic critical moment for lateral torsional buckling |
| *M*c,Rd | design value of the bending moment resistance about one principal axis of a cross-section |
| *M*Ed | design value of the bending moment |
| *M*rw,max | calculated bending moment due to rain-wind-induced vibration |
| *M*w,max | calculated bending moment |
| *M*1*, M*2 | end moments |
| *N*E | reference elastic critical axial force |
| *N*Ed | design value of the axial force |
| *N*cr | elastic critical axial force for the relevant buckling mode based on the gross cross-sectional properties |
| *N*c,Rd | design value of the resistance to axial force of the cross-section for uniform compression |
| *N*Ed | design value of the axial force |
| *N*i | number of lorries per year in lane i |
| *N*obs | total number of lorries per year in the slow lane |
| *N*0 | reference number of cycles |
| *Q*mi | average gross weight of the lorries in lane i |
| *Q*i | gross weight of the lorry i |
| *Q*0 | reference weight |
| *R* | radius of cope hole |
| *R*B | radius of the beam in plan |
| *R*eH | yield strength taken from the relevant product standard |
| *R*m | ultimate strength taken from the relevant product standard |
| *St* | Strouhal number |
| *V*b,Rd | design value of the shear buckling resistance |
| *V*bw,Rd | design value of the shear buckling resistance considering the contribution of the web |
| *V*Ed | design value of the shear force |
| *W*el | elastic section modulus |
| *W*el,com | elastic modulus which corresponds to the fibre where the limiting compressive stress is obtained |
| *W*el,min | elastic modulus which corresponds to the fibre with the maximum stress (tension or compression). |
| *Z*Ed | required design Z-value resulting from the magnitude of strains from restrained metal shrinkage under the weld beads |

### Latin lower-case symbols

|  |  |
| --- | --- |
| *a*R | spacing of the rigid transverse stiffeners |
| *a*σ | ratio of stress ranges |
| *a* | weld size |
| *a*0, *b*0, *c*0 | stability values for galloping oscillations in the bending mode |
| *b*c | width of the compression flange |
| *b* | plate width |
| *b*eff | effective width |
| *b*F | width at cope hole level |
| *b*p | width of unstiffened panel |
| *b*st | width of stiffener |
| *b*u | maximum width of plate |
| *b*w | width of weld |
| *b*1c, *b*2c | width of a crossbeam submitted to local load from stiffeners |
| *c* | stiffness parameter |
| *c* | exciting coefficient |
| *c*lat | exciting power coefficient |
| *d* | plate thickness of flat steel plate hanger |
| *d*c | distance between adjacent crossbeams |
| *d*s | distance between springs |
| *e* | spacing of the supports of the deck plate |
| *e*crossb | spacing of crossbeams |
| *e*E | edge distance of crossbeams |
| *e*LS | spacing between stiffeners |
| *e*Q, *e*s | eccentricities in a forged hanger |
| *e*0 | equivalent geometrical imperfection |
| *f* | natural frequency |
| *f*i | natural frequency of the relevant mode |
| *f*u | ultimate tensile strength |
| *f*ub | nominal tensile strength of bolts |
| *f*y | yield strength |
| *f*yb | nominal yield strength for bolts |
| *f*yw | yield strength of the web |
| *f*0 | reference frequency |
| *h*crossb | height of crossbeam |
| *h* | height of stiffener |
| *h*stiff | height of stiffener |
| *h*w | height of the web |
| *k* | number of lanes with heavy traffic |
| *k*d | factor to take account of the continuous reduction of the excitation risk due to rain-wind induced vibrations |
| *k*F,i | factor to take account of a continuous reduction in the level of excitation as the natural frequencies increase |
| *k*H,i | reduction factor to take account of frequency |
| *k*s | stress modification factor |
| *k*T,i | reduction factor to take account of turbulence |
| *k*V,i | factor to take account of the continuous decrease in the excitation at wind velocities in excess of 20 m/s |
| *k*α | amplification factor for taking into account second order sway effects |
| *k*τ,za | critical shear buckling factor for the web of a curved girder |
| *m* | amplification factor applied to the reference elastic critical axial force |
| *m*L | mass per unit length |
| *n* | percentage of traffic |
| *n*i | number of lorries of gross weight *Q*i in the slow lane |
| *q* | lateral load for the effects of oscillations due to vortex shedding |
| *q*max | maximum value of the equivalent lateral load |
| *q*red | reduced value of the equivalent lateral load for the effects of oscillations due to vortex shedding |
| *q*0 | reference value of the lateral load |
| *r* | radius |
| *r*g | root gap or root reset |
| *r*u, *r*1 | radius in a cope hole |
| *s* | length of a arch |
| *s* | gap width |
| *t* | thickness |
| *t*D | thickness of deck plate |
| *t*f,cb | flange thickness of crossbeam |
| *t*stiff | thickness of stiffener |
| *t*Ld | design service life of the bridge in years |
| *t*w | web thickness |
| *t*w,cb | web thickness of a crossbeam |
| *t*w,st | plate thickness of stiffeners |
| *v* | imperfection of forged hanger end |
| *v*cr,i | critical wind velocity of the relevant mode |
| *V*m | mean value of the wind velocity at mid height of the hangers |
| *v*0 | reference value of the critical wind velocity |
| *z*a | curvature parameter for shear resistance |
| *z*f | curvature parameter for bending resistance |

### Greek upper-case symbols

|  |  |
| --- | --- |
| *Φ* | non dimensional factor depending on ratios of shear forces and bending moments |
| *Φ*LT | value to determine the reduction factor for lateral torsional buckling |
| *Φ*R | Diameter of rope hangers |
| *Φ*2 | damage equivalent impact factor |
| Δ*b*1, Δ*b*2 | imperfections in forged hanger end |
| Δ*σ* | stress range |
| Δ*σ*c | characteristic reference value of fatigue resistance in terms of normal stress, at 2 × 106 cycles |
| Δ*σ*e,2 | damage equivalent normal stress range related to 2 × 106 cycles |
| Δ*σ*fre | nominal stress range due to the frequent load combination |
| Δ*σ*p, Δ*σ*loc, Δ*σ*glo | stress ranges from load *p* |
| Δ*σ*rw | stress range due to rain-wind-induced vibrations |
| Δ*σ*w,e,2 | stress range at the points likely to be affected by fatigue |
| Δ*σ*1 | stress range at the section due to load model 71 on one track |
| Δ*σ*1+2 | stress range at the section to be checked due to load model 71 on any two tracks |
| Δ*τ*e,2 | damage equivalent shear stress range related to 2 × 106 cycles |
| Δ*τ*c | characteristic reference value of fatigue resistance in terms of shear stress, at 2 × 106 cycles |
| *Θ* | mass moment of inertia |
| *Φ*glo | damage equivalent impact factor for global effects |
| *Φ*loc | damage equivalent impact factor for local effects |
| *Φ*2 | damage equivalent impact factor |

### Greek lower-case symbols

|  |  |
| --- | --- |
| *α* | angle between flange and web |
| *α*cr | factor by which the design loading would have to be increased to cause elastic instability in a global mode |
| *α*m | reduction factor for the number of piers in a row |
| *α*w | aspect ratio of a web panel |
| *δ* | logarithmic decrement of damping |
| *δ*meas | logarithmic decrement of damping obtained by measurements |
| *δ*T | logarithmic decrement of damping of the first torsional mode |
| *ε* | material parameter depending on the yield strength |
| *γ* | stiffness ratio |
| *γ*Ff | partial factor for the fatigue action effect |
| *γ*Mf | partial factor for the fatigue resistance |
| *γ*M,ser | partial factor for serviceability limit states |
| *γ*M0 | partial factor for resistance of cross-sections |
| *γ*M1 | partial factor for resistance of members to instability assessed by member checks |
| *γ*M2 | partial factor for resistance of cross-sections in tension to fracture, for resistance of bolts, for resistance of rivets, for resistance of pins, for resistance of welds for resistance of plates in bearing |
| *γ*M3 | partial factor for slip resistance of joints at ultimate limit state (Category C) |
| *γ*M3,ser | partial factor for slip resistance of joints at serviceability limit state |
| *γ*M4 | partial factor for bearing resistance in an injection bolt |
| *γ*M5 | partial factor for resistance of joints in hollow section lattice girders |
| *γ*M6,ser | partial factor for resistance of pins at serviceability limit states |
| *γ*M7 | partial factor for preload of high strength bolts |
| *γ*c | partial factor for resistance of concrete |
|  | relative stiffness of a longitudinally stiffened web panel |
| *η*i | value of the influence line for the internal force that produces the stress range in the middle of lane i |
| *λ*glo | damage equivalent factor for global effects |
| *λ*loc | damage equivalent factor for local effects |
| *λ*, *λ*1, *λ*2, *λ*3, *λ*4 | damage equivalent factors |
|  | relative slenderness for lateral torsional buckling |
|  | relative slenderness for shear buckling of the web |
|  | limit of relative slenderness for susceptibility to shear buckling of the web |
| *λ*max | maximum λ-value taking account of the fatigue limit |
|  | maximum λ-value taking account of the fatigue limit |
| *μ* | ratio of shear forces |
| *ρ* | density of the air |
| *ρ*c,x | reduction factor |
| *σ*cr,p | longitudinal critical stress of a web panel or sub-panel |
| *σ*Ed | design value of a normal stress |
| *σ*Ed,ser | normal stress at serviceability limit states |
| *σ*p,max | maximum normal stress for the calculation of the reference stress range |
| *σ*p,min | minimum normal stress for the calculation of the reference stress range |
| *τ*Ed,ser | shear stress at serviceability limit states |
| *σ*G | stress due to the axial force acting on the hangers under permanent actions |
| *σ*glob,Ed | design value of stress in the stiffener due to bridge loads comprising one or more heavy vehicles |
| *σ*limit | limiting stress of the weakest part of the cross-section in compression |
| *σ*loc,Ed | design value of stress in a stiffener due to local wheel or tyre load from a single heavy vehicle |
| *σ*net,Rd | design value of the tensile strength at cope hole level |
| *σ*p,max | maximum stress |
| *σ*p,min | minimum stress |
| *σ*Q | stress due to the axial force and bending moment of the hanger as a result of frequent traffic loads |
| *σ*Rd | design value of the tensile strength of a hanger |
| *σ*rw | maximum stress amplitude due to rain-wind-induced vibration |
| *σ*x,Ed | design value of longitudinal stress |
| *σ*x,Ed,ser | longitudinal stress at serviceability limit states |
| *σ*1, *σ*2 | direct stresses |
| *σ*1b, *σ*2b | stresses due to bending |
| *σ*1c, *σ*2c | compressive stresses due to local load from stiffeners |
| *τ*cr | elastic critical shear stress |
| *τ*Rd | design value of the shear strength at cope hole level |
| *τ*Ed,ser | shear stress at serviceability limit states |
| *Φ*A, *Φ*B, *Φ*R, *Φ*H | diameters of hanger components |
| *Φ*s, *Φ*u, *Φ*0 | angles of imperfection of forged hanger end |
| *χ*w | reduction factor for shear buckling |
| *ψ* | combination factor |

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

# Basis of design

## General rules

### Basic requirements

(1) The design of steel bridges and steel parts of steel-concrete composite bridges shall be in accordance with the general rules given in EN 1990, EN 1991 (all parts) and EN 1998 (all parts) and the specific design provisions for steel structures given in EN 1993‑1 (all parts).

(2) Steel bridges and steel parts of steel-concrete composite bridges designed according to this document shall be executed according to EN 1090‑2 and EN 1090‑4, with construction materials and products used as specified in the relevant parts of EN 1993, or in the relevant material and product specifications.

NOTE Additional guidance is given in Annex C for orthotropic decks.

### Structural reliability

(1) The rules in EN 1993‑1‑1 apply.

(2) The choice of the execution classes may be specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

### Robustness

(1) EN 1993‑1‑1:2022, 4.1.3, applies.

(2) For robustness, 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.

NOTE 1 EN 1990 provides additional information about accidental design situations and robustness criteria.

NOTE 2 The National Annex can define components that are subject to accidental design situations and also details for assessments. Examples of such components are hangers, tension components, bearings.

### Design service life for bridges

(1) The design service life of bridges should be taken in accordance with EN 1990:2023, Table A.2.2.

(2) The required design service life should be achieved through design for fatigue, see Clause 10, and/or appropriate detailing, see Annex C, and by serviceability checks, see Clause 9.

(3) For structural elements that cannot be designed for the total design service life of the bridge, 6(5) and 6(6) should be considered.

### Durability

(1) EN 1993‑1‑1:2022, 4.1.5, applies.

(2) For durability, see Clause 6.

## Basic variables

### Actions and environmental influences

(1) EN 1993‑1‑1:2022, 4.3.1, applies.

(2) Actions for the design of bridges should be taken from EN 1991 (all parts). For the combination of actions and partial factors for actions, see EN 1990:2023, A.2.

(3) For the determination of combinations of effects from local and global loading on steel bridge decks of road bridges use Annex E.

(4) For actions not defined in EN 1991 (all parts), additional rules may be specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE For actions on bearings, see EN 1990.

### Material and product properties

(1) See Clause 5 and EN 1993‑1‑1.

## Verification by the partial factor method

(1) EN 1993‑1‑1:2022, 4.4, applies.

(2) The partial factors *γ*Mi in Table 4.1 should be applied to the characteristic values of resistance in ultimate and serviceability limit states.

Table 4.1 — Partial factors

|  |  |
| --- | --- |
| 1. 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 |
| 1. resistance of joints | |
| * resistance of bolts | *γ*M2 |
| * resistance of rivets | *γ*M2 |
| * resistance of pins | *γ*M2 |
| * resistance of welds | *γ*M2 |
| * resistance of plates in bearing | *γ*M2 |
| * slip resistance |  |
| * at ultimate limit state (Category C) | *γ*M3 |
| * at serviceability limit state | *γ*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 | *γ*M6,ser |
| * partial factor for resistance of components when verifying structural integrity (tying resistance) | *γ*Mu |
| * resistance of tension components | see prEN 1993‑1‑11:2024 |
| * resistance of concrete | *γ*c  see EN 1992 (all parts) |
| 1. Serviceability | |
| * stress limitation (see 9.3) | *γ*M,ser |

NOTE 1 The numerical values for partial factors *γ*Mi for bridges for non-accidental design situations are the following, unless the National Annex gives different values:

* *γ*M0 = 1,00;
* *γ*M1 = 1,10;
* *γ*M2 = 1,25;
* *γ*M3 = 1,25;
* *γ*M3,ser = 1,10;
* *γ*M4 = 1,10;
* *γ*M5 = 1,10;
* *γ*M6,ser = 1,00;
* *γ*Mu = 1,10;
* *γ*M,ser = 1,00.

NOTE 2 The numerical values for partial factors *γ*Mi for bridges for accidental design situations are the following, unless the National Annex gives different values:

* *γ*M0 = 1,00;
* *γ*M1 = 1,00;
* *γ*M2 = 1,15.

For all other partial factors, the values are as given in Note 1, unless the National Annex gives different values.

## Partial factors for fatigue verifications

(1) The partial factor for fatigue loads should be taken as *γ*Ff .

NOTE The value of the partial factor is *γ*Ff = 1,0 according to EN 1990:2023, A.2 unless the National Annex gives a different value.

(2) The partial factor for fatigue resistance should be taken as *γ*Mf .

NOTE The values of the partial factor *γ*Mf are given in EN 1993‑1‑9 unless the National Annex gives different values.

## Design assisted by testing

(1) EN 1993‑1‑1:2022, 4.5 applies.

(2) 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 provide sufficient assurance of the structural safety during either the erection stage or the design service life.

NOTE prEN 1991‑1‑4:2024, Annex K gives guidance on derivation of design parameters from wind tunnel tests and numerical simulations.

# Materials

## General

(1) EN 1993‑1‑1:2022, 5.1 applies.

## Structural steel

### Material properties

(1) The nominal values of the yield strength *f*y and the ultimate strength *f*u for structural steel should be obtained:

1. either by adopting the values *f*y = *R*eH and *f*u = *R*m (as lower bound of the given range) directly from the product standard;
2. or by using the values given in EN 1993‑1‑1:2022, Table 5.1 for steel conforming to EN 10025 (all parts), EN 10210 (all parts), EN 10219 (all parts), and in EN 1993‑1‑1:2022, Table 5.2 for steel conforming to EN 10149 (all parts) and considering the availability of the material in the thickness range according to the product standard.

NOTE 1 The choice for option a) or b) can be set by the National Annex, considering the effects on partial factors (see Table 4.1) and their calibration according to EN 1993‑1‑1:2022, Annex E and EN 1990.

NOTE 2 Rules for the use of the steels according to EN 1993‑1‑1:2022, Table 5.1 and Table 5.2 can be set by the National Annex.

### Ductility requirements

(1) EN 1993‑1‑1:2022, 5.2.2 applies.

### Fracture toughness

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

(2) The rules in EN 1993‑1‑10 for fracture toughness apply.

NOTE The National Annex can specify conditions of application of EN 1993‑1‑10 to bridges and can give additional requirements.

### Through thickness properties

(1) Steel with improved through thickness properties conforming 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 is given in Table 5.2, unless the National Annex gives different values.

Table 5.2 (NDP) — Quality class conforming to EN 10164

| Target value *Z*Ed | Quality class |
| --- | --- |
| *Z*Ed ≤ 10 | — |
| 10 < *Z*Ed ≤ 20 | Z15 |
| 20 < *Z*Ed ≤ 30 | Z25 |
| *Z*Ed > 30 | Z35 |

### Values of other material properties

(1) EN 1993‑1‑1:2022, 5.2.5, applies.

## Connecting devices

### Fasteners

#### Bolts, nuts and washers

(1) Bolts, nuts and washers should conform to the rules given in FprEN 1993‑1‑8:2023, 5.1.

(2) Bolt grades 4.8 and 5.8 should not be used in bridges.

#### Rivets

(1) Rivets should conform to the rules given in FprEN 1993‑1‑8:2023, 5.2.

NOTE The material properties, dimensions and tolerances of steel rivets can be given in the National Annex.

#### Anchor bolts

(1) Anchor bolts should conform to the rules given in FprEN 1993‑1‑8:2023, 5.3.

### Welding consumables

(1) All welding consumables should conform to the rules given in FprEN 1993‑1‑8:2023, 6.2.

(2) The filler metal should be selected according to the rules given in EN 1993‑1‑8.

NOTE Restrictions to the use of undermatched filler metals can be set by the National Annex.

## Cables and other tension elements

(1) Cables and other tension elements should be designed according to EN 1993‑1‑11.

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

(2) Use the additional provisions according to Annex A for certain types of hangers in tied-arch bridges.

## Bearings

(1) Bearings should conform to EN 1337 (all parts).

NOTE The National Annex can give requirements for the installation and the use of bearings applicable to bridges.

(2) Bearings should be designed according to EN 1990:2023, Annex G.

## Dampers and lock-up devices

(1) Dampers and lock-up devices should conform to EN 15129 and to the relevant technical specifications.

NOTE The National Annex can give requirements for the installation and the use of dampers and lock-up devices applicable to bridges.

## Other bridge components

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

NOTE The National Annex can give requirements for the installation and the use of expansion joints, guardrails, parapets and other ancillary items applicable to bridges.

(2) Expansion joints should be designed according to EN 1990:2023, A.2.10.4.

(3) The bridge deck surfacing system, the products used and the method of application should conform to the relevant technical specification.

NOTE The National Annex can give requirements for the bridge deck surfacing system, the products used and the method of application for bridges.

# Durability

(1) EN 1993‑1‑1:2022, Clause 6, applies.

(2) The effects of corrosion or fatigue of components and material should be taken into account by appropriate detailing. The effects of fatigue should be taken into account in accordance with Clause 10, EN 1993‑1‑9 and EN 1993‑1‑10.

(3) Elements or parts of elements that cannot be inspected should be avoided where possible. Where they are unavoidable, appropriate corrosion allowances should be provided.

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

(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.

NOTE Injection bolts can also be used if permitted by 11.1.2.

(5) For components, such as tension components, bearings and expansion joints, that cannot be designed with sufficient reliability to achieve the design service life of the bridge, the rules in EN 1990:2023, A.2.3(2) and (3) apply.

(6) Structural parts of a bridge to which guardrails or parapets are connected, should be designed according to EN 1990:2023, A.2.6.8(3).

(7) The design service life of the bridge and its components according to 4.1.4 should be attained by:

* fatigue design of details in accordance with (2), (3) and EN 1993‑1‑9 and with serviceability checks carried out in accordance with Clause 9;
* appropriate structural detailing (e.g. orthotropic decks);
* material chosen in accordance with Clause 5;
* fabrication conforming to EN 1090‑2 and EN 1090‑4.

NOTE Recommendations for structural detailing of orthotropic steel decks are given in Annex C. Alternatively, orthotropic steel decks can be designed according to CEN/TS 1993‑1‑901[[2]](#footnote-2). The National Annex can provide additional information on the use of these two possibilities.

# Structural analysis

## Structural modelling for analysis

### Basic assumptions

(1) EN 1993‑1‑1:2022, 7.1.1, applies.

(2) For the analysis of cable supported structures, EN 1993‑1‑11 applies.

### Joint modelling

(1) EN 1993‑1‑1:2022, 7.1.2 applies.

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

NOTE Fatigue categories given in EN 1993‑1‑9 are appropriate for rigid joints between members of bridges except for bearings, pinned connections and cables.

## Global analysis

### Consideration of second order effects

(1) EN 1993‑1‑1:2022, 7.2.1, applies assuming the rules for sway modes apply to global modes for bridges.

### Methods of analysis for ultimate limit state design checks

(1) EN 1993‑1‑1:2022, 7.2.2, applies.

(2) Where the behaviour of a bridge or its components is governed by the first buckling mode, on the basis of elastic global analysis, the second order bending moment *M*II due to the applied axial force may be calculated by increasing the imperfections and other deformations according to the first order theory by the factor *k*α calculated from the Formula (7.1):

|  |  |  |
| --- | --- | --- |
|  | provided that: | (7.1) |

where

|  |  |
| --- | --- |
|  | is the factor by which the design loading would have to be increased to cause elastic instability in a global mode. |

(3) For *α*cr < 3, a more accurate second order analysis should be applied.

(4) For *α*cr ≥ 10, the second order effects may be neglected, see EN 1993‑1‑1.

## Imperfections

### Basis

(1) EN 1993‑1‑1:2022, 7.3.1, applies.

### Sway imperfections

(1) EN 1993‑1‑1:2022, 7.3.2, applies.

NOTE For piers, αm is applicable if cumulative effects from contributions of various piers occur (e.g. for piers forming a frame with the superstructure).

(2) For arched bridges, use the rules in Annex D.

### Equivalent bow imperfection for global and member analysis

#### Flexural buckling

(1) EN 1993‑1‑1:2022, 7.3.3.1 applies.

#### Lateral torsional buckling

(1) EN 1993‑1‑1:2022, 7.3.3.2 applies.

### Combination of sway and equivalent bow imperfections

(1) EN 1993‑1‑1:2022, 7.3.4 applies.

### Imperfections for analysis of bracing systems

(1) EN 1993‑1‑1:2022, 7.3.5 applies.

### Imperfection based on elastic critical buckling modes

(1) EN 1993‑1‑1:2022, 7.3.6 applies.

## Methods of analysis

### General

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

(2) Plastic global analysis according to EN 1993‑1‑1:2022, 7.4.3, may be used for accidental design situations.

NOTE The National Annex can give restrictions on the use of plastic global analysis.

(3) If a finite element model (FEM) analysis is used, EN 1993‑1‑14 should be applied.

### Elastic global analysis

(1) EN 1993‑1‑1:2022, 7.4.2, applies.

(2) If all cross-sections are class 1 and member resistance is not reduced by buckling, the effects of differential temperature, shrinkage and settlement at the ultimate limit state may be neglected.

## Classification of cross-sections

(1) EN 1993‑1‑1:2022, 7.5, applies.

# Ultimate limit states

## Partial factors

(1) The partial factors on resistance should be taken from 4.3.

## Resistance of cross-sections

### General

(1) EN 1993‑1‑1:2022, 8.2.1, applies.

### Section properties

#### Gross cross-section

(1) EN 1993‑1‑1:2022, 8.2.2.1, applies.

#### Net area

(1) EN 1993‑1‑1:2022, 8.2.2.2, applies.

#### Shear lag effects

(1) EN 1993‑1‑1:2022, 8.2.2.3, and EN 1993-1-5 apply.

NOTE The National Annex can give restrictions on the method to consider shear lag effects at the ultimate limit state.

#### Effective properties of cross-section with class 3 webs and class 1 or 2 flanges

(1) EN 1993‑1‑1:2022, 8.2.2.4 applies.

#### Effective cross-section properties of class 4 cross-sections

(1) The effects of local plate 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
2. limiting the stress levels on the gross cross-section in accordance with FprEN 1993-1-5:2023, Clause 12.

NOTE The National Annex can specify the methods to be used. In case of the use of the method b), the National Annex can give further guidance.

(2) For class 4 circular hollow sections, the effects of local buckling should be considered using one of the following two methods:

1. effective cross-section properties of class 4 sections in accordance with EN 1993‑1‑1
2. limiting the stress levels on the gross cross-section in accordance with EN 1993‑1‑6.

#### Section properties for the characteristic resistance

(1) EN 1993‑1‑1:2022, 8.2.2.6 applies.

### Tension

(1) EN 1993‑1‑1:2022, 8.2.3 applies.

(2) For the design of steel tension components comprising rods, ropes and bundles of wires, EN 1993‑1‑11 applies.

(3) For certain types of hangers in tied-arch bridges, additional rules according to Annex A should apply.

### Compression

(1) EN 1993‑1‑1:2022, 8.2.4 applies.

(2) As an alternative for class 4 cross-sections, the design resistance may be calculated using Formula (8.1):

|  |  |
| --- | --- |
|  | (8.1) |

where

|  |  |
| --- | --- |
| *σ*limit = *ρ*c,x *f*y | is the limiting stress of the weakest part of the cross-section in compression |
| *ρ*c,x | is the reduction factor according to FprEN 1993-1-5:2023, 12.4. |

### Bending

(1) EN 1993‑1‑1:2022, 8.2.5 applies.

(2) As an alternative for class 4 cross-sections, the design resistance may be calculated using Formula (8.2):

|  |  |
| --- | --- |
|  | (8.2) |

where

|  |  |
| --- | --- |
| *σ*limit = *ρ*c,x *f*y | is the limiting compressive stress of the weakest part of the cross-section; |
| *ρ*c,x | is the reduction factor according to FprEN 1993-1-5:2023, 12.4; |
| *W*el,com | is the elastic modulus which corresponds to the fibre where the limiting compressive stress is obtained; |
| *W*el,min | is the elastic modulus which corresponds to the fibre with the maximum stress (tension or compression). |

### Shear

(1) EN 1993‑1‑1:2022, 8.2.6 and EN 1993-1-5 apply.

### Torsion

#### General

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

(2) An appropriate elastic model (such as a grillage model, a shell model, a higher-order beam model or a beam on elastic foundation model), taking into account the member cross-section and the stiffness and the location of any diaphragms or cross-bracings present, should be used to determine the combined effect of bending, torsion and distortion.

(3) Distortional effects in box-girder members may be disregarded where the longitudinal stresses at any cross-section created by distortion under the governing eccentrical ULS combination of actions do not exceed either 10 % of the longitudinal stresses caused by the longitudinal bending moment or 0,05 *f*yd, whichever is greater, due to the same combination of actions and at the same cross-section.

(4) Diaphragms or cross-bracings should be designed taking into account the torsional and distortional effects.

#### Torsion for which distortional effects may be neglected

(1) EN 1993‑1‑1:2022, 8.2.7 applies.

### Combined bending and shear

(1) EN 1993‑1‑1:2022, 8.2.8 applies.

### Combined bending and axial force

#### Class 1 and class 2 cross-sections

(1) EN 1993‑1‑1:2022, 8.2.9.1 applies.

#### Class 3 cross-sections

(1) EN 1993‑1‑1:2022, 8.2.9.2 applies.

#### Class 4 cross-sections

(1) EN 1993‑1‑1:2022, 8.2.9.3 applies.

(2) When using the limiting stress method according to 8.2.4(2) or 8.2.5(2), the criterion in Formula (8.3) should be met:

|  |  |
| --- | --- |
|  | (8.3) |

where *σ*limit should be determined from 8.2.4 or 8.2.5.

### Combined bending, shear and axial force

(1) EN 1993‑1‑1:2022, 8.2.10 applies.

### Combined bending, shear, axial force and transverse loads

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

1. Interaction methods given in 8.2.8 to 8.2.10.

NOTE For plate buckling effects, see FprEN 1993-1-5:2023, Clause 6 to Clause 9.

1. Interaction of stresses using the yielding criterion given in 8.2.1

NOTE For plate buckling effects, see FprEN 1993-1-5:2023, Clause 12.

## Buckling resistance of members

### Uniform members in compression

#### Buckling resistance

(1) EN 1993‑1‑1:2022, 8.3.1.1 applies.

#### Slenderness of compression members

(1) EN 1993‑1‑1:2022, 8.3.1.2 applies.

#### Buckling reduction factor for flexural buckling

(1) EN 1993‑1‑1:2022, 8.3.1.3 applies.

#### Buckling reduction factors for torsional and torsional-flexural buckling

(1) EN 1993‑1‑1:2022, 8.3.1.4 applies.

#### Use of class 3 section properties with stress limits

(1) As an alternative to using class 4 cross-section properties, class 3 cross-section properties may be used in conjunction with stress limits in accordance with FprEN 1993-1-5:2023, Clause 12.

### Uniform members in bending

(1) EN 1993‑1‑1:2022, 8.3.2 applies.

### Uniform members in bending and axial compression

(1) EN 1993‑1‑1:2022, 8.3.3 applies.

(2) As an alternative to Formulae (8.88) and (8.89) in EN 1993‑1‑1:2022, 8.3.3, the simplified method of CEN/TS 1993‑1‑101 may be used.

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

(1) EN 1993‑1‑1:2022, 8.3.4, applies.

### Simplified method for lateral and lateral torsional buckling of structural components

(1) 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 CEN/TR 1993‑1‑103[[3]](#footnote-3).

NOTE 2 Where truss flanges and/or chords 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 or chords.

NOTE 3 This method cannot verify the resistance of beams braced together where a mode of overall buckling involving two or more beams can occur, such as the combined rotation of paired beams in the absence of plan bracing or a deck plate or slab. In such cases, the elastic critical bending moment and slenderness for buckling is obtained from an elastic critical buckling analysis or from literature.

(2) 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 buckling length used to calculate the elastic critical buckling should not be less than the spacing of the restraints.

(3) The verification to lateral torsional buckling may be carried out in accordance with 8.3.2 using Formula (8.4):

|  |  |
| --- | --- |
|  | (8.4) |

where

|  |  |
| --- | --- |
| *A*eff | is the effective area of the chord; |
| *N*cr | is the elastic critical load determined with the gross cross-section of the chord. |

(4) 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 a first order calculation, by applying an additional lateral force *F*Ed at the connection of the chord to the spring according to the Formulae (8.5):

|  |  |
| --- | --- |
|  | (8.5) |

where

|  |  |
| --- | --- |
| *L*w | is the half wavelength of buckling determined by taking *L*R/*L*w as the next integer below *L*R/*L*crbut not less than unity; |
| *L*R | is the span length between the rigid supports, for example by rigid cross-frames; |
| *d*s | is the distance between the springs; |
| *C*d | is the spring stiffness, see (1), NOTE 1. |

(5) If the design value of the compression force *N*Ed is constant over the length of the chord, the critical axial load *N*cr may be calculated using Formula (8.6):

|  |  |
| --- | --- |
|  | (8.6) |

where

A lateral support to a compressed flange may be assumed to be rigid if its stiffness *C*d satisfies the condition in Formula (8.7):

|  |  |
| --- | --- |
|  | (8.7) |

where *N*E is the elastic critical load which is determined assuming hinged ends. If there are no rigid lateral supports at girder ends to define the length *L*R*,* either the slenderness should be determined in accordance with (2) or the value of m should be modified in the last half wavelength of buckling to allow for the end restraint flexibility in accordance with elastic buckling theory.

(6) The procedure given in (1) to (5) may also be applied to the flanges of girders in compression when *A*eff in (3) is substituted by the Formula (8.8):

|  |  |
| --- | --- |
|  | (8.8) |

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.

For the case where the compressive force *N*Ed is not constant over the length of the chord, the approach in (7) and (8) is recommended.

NOTE The National Annex can give further guidance or a different approach for the case where the compressive force *N*Ed is not constant over the length of the chord.

(7) For the bottom flange of a continuous girder with rigid lateral supports at a distance *L*R (see Figure 8.1), m in Formula (8.6) may be taken as the minimum value obtained from the Formulae (8.9) and (8.10):

|  |  |
| --- | --- |
|  | (8.9) |
| *m* = 1 + 0,44 (1 + *μ*) *Φ*1,5 + (0,195 + (0,05 + *μ*/100) *Φ*) *γ*0,5 | (8.10) |

where

|  |  |
| --- | --- |
|  | see Figure 8.1 |
|  | for *M*2 > 0 |

Where the bending moment changes signs, Formulae (8.9) and (8.10) may be used as a conservative estimate by inserting *M*2 = 0.

(8) The verification of resistance to lateral torsional buckling in accordance with 8.3.2 may be carried out at a distance 0,25 *L*cr from the support with the largest moment as shown in Figure 8.1, provided that the cross-sectional resistance is also checked at the section with the largest moment, where .



Key

|  |  |
| --- | --- |
| 1 | design section |

Figure 8.1 — Segment of beam between rigid lateral supports with bending moment varying as a parabola

### Plate girders curved in plan

(1) The effects of any curvature in plan should be considered on the resistances of plate girders.

(2) Unless an advanced approach considering the nonlinear behaviour directly is used, use the rules of Annex B.

## Uniform built-up compression members

(1) EN 1993‑1‑1:2022, 8.4 applies.

## 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:

1. Direct stresses, shear stresses and transverse forces should be verified according to FprEN 1993-1-5:2023, Clause 4, Clause 5 and Clause 6. Additionally, the interaction criteria in FprEN 1993-1-5:2023, Clause 9, should also be met.
2. Reduced stress method on the basis of stress limits governed by plate buckling according to FprEN 1993-1-5:2023, Clause 12.

NOTE See also 8.2.2.5.

(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 8.3.3. For stiffened deck plates, the effective cross-section for the verification should allow for local plate buckling in accordance with EN 1993-1-5 and the reduction factor for out of plane buckling in EN 1993‑1‑1:2022, 8.3.1.3, should be modified in accordance with FprEN 1993-1-5:2023, 6.5.3(5).

NOTE This method assumes that global buckling is column-like as a conservative simplification.

# Serviceability limit states

## General

(1) EN 1993‑1‑1:2022, 9.1 applies.

(2) The following serviceability criteria should be met:

1. Restriction to elastic behaviour in order to limit:

* excessive yielding, see 9.3(1);
* deviations from the intended geometry by residual deflections, see 9.3(1);
* excessive deformations, see 9.3(4).

1. Limitation of deflections and curvature in order to prevent:

* infringement of required clearances, see 9.5 or 9.6;
* unacceptable changes in railway track geometry and excessive rail stresses, see 9.7;
* unwanted dynamic impacts due to traffic (combination of deflection and natural frequency limitations), see 9.7 and 9.8;
* cracking of surfacing layers, see 9.8;
* damage of drainage, see 9.12.

1. Limitation of natural frequencies in order to:

* exclude vibrations due to traffic or wind which are unacceptable to pedestrians or passengers in cars using the bridge, see 9.8 and 9.9;
* limit fatigue damages caused by resonance, see 9.7, 9.8 and 9.9;
* limit excessive noise emission, see 9.8 and 9.9;
* prevent ballast instability and unacceptable reduction in wheel rail contact forces for railway vehicles, see 9.7.

1. Restriction of plate slenderness, see 9.4, 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.

1. Improved durability by appropriate detailing to reduce corrosion and excessive wear, see 9.11.
2. Ease of maintenance and repair, see 9.11, 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.

(3) 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 can give guidance on serviceability requirements for specific types of bridges.

## 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.

(4) EN 1993‑1‑14 should be used for calculation models based on the finite element method.

## Limitations for stress

(1) The nominal stresses *σ*Ed,ser and *τ*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 according to the criteria in Formulae (9.1), (9.2) and (9.3):

|  |  |
| --- | --- |
|  | (9.1) |
|  | (9.2) |
|  | (9.3) |

Where relevant, the above checks should include stresses *σ*z from transverse loads, see EN 1993-1-5.

NOTE 1 *γ*M,ser. *γ*M,ser = 1,00, unless the National Annex gives a different value.

NOTE 2 For global analysis, the effects of plate buckling on the stiffness can be ignored as specified in FprEN 1993-1-5:2023, 4.3(9).

(2) The nominal stress range Δ*σ*fre, due to the frequent load combination should be limited to 1,5 *f*y/*γ*M,ser.

NOTE This verification is meant to avoid low cycle fatigue, see EN 1993‑1‑9.

(3) For non-preloaded bolted connections subject to shear, the bolt shear force, *F*v,Ed,ser, due to the characteristic load combination should be limited according to condition in Formula (9.4):

|  |  |
| --- | --- |
|  | (9.4) |

where *F*b,Rd is the design 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.

## 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 can define cases where web breathing checks are not necessary.

(2) Web breathing may be neglected for unstiffened panels where the conditions in Formula (9.5) or (9.6) are met:

|  |  |  |
| --- | --- | --- |
|  | for road bridges | (9.5) |
|  | for railway bridges | (9.6) |

where

|  |  |
| --- | --- |
| *b*p | is the width of the panel |
| *t* | is the thickness of the panel |
| *L* | is the span length in metres, but not less than 20 m. |

(3) Web breathing may be neglected for longitudinally stiffened web panels with a relative stiffness according to FprEN 1993-1-5:2023, 6.5.1(5) greater than 25, where the conditions in Formula (9.5) or (9.6) are met for each subpanel, assuming *b* as the width of the subpanel.

(4) If the provisions in (2) or (3) are not satisfied, web breathing should be checked using the criterion in Formula (9.7):

|  |  |
| --- | --- |
|  | (9.7) |

where

|  |  |
| --- | --- |
| *σ*x,Ed,ser, *τ* Ed,ser | are the stresses for the frequent load combination. If the stresses are not uniform along the length *a* of the panel, FprEN 1993-1-5:2023, 6.7(4) should be considered. The stress *σ*x,Ed,ser should be taken as the stress at the compressive edge of the panel or subpanel being checked. For panels or subpanels wholly in tension, *σ*x,Ed,ser/σcr,p may be taken equal to zero. |
| *σ*cr,p and *τ*cr | are the critical buckling stresses of the panel or subpanel according to EN 1993-1-5. If relevant, *σ*cr,p should take the column type buckling behaviour of the panel into account. |

(5) For webs with longitudinal stiffeners not satisfying 9.4(3), either 9.4(4) should be applied to each subpanel and to the stiffened panel or 9.4(2) should be applied to the overall panel, neglecting the longitudinal stiffeners.

## Limits for clearance gauges

(1) EN 1990:2023, A.2.8.1(4) applies.

## Limits for visual impression

(1) To achieve a satisfactory appearance of the bridge, consideration should be given to precambering on a project-specific basis.

(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.

## Performance criteria for railway bridges

(1) Specific criteria for deflection and vibrations for railway bridges should be obtained from EN 1990:2023, A2.

(2) Any requirements for the limitation of noise emission may be specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

## Performance criteria for road bridges

### General

(1) EN 1990:2023, A.2.8.2(1) applies.

### 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.

(2) EN 1990:2023, A.2.10.4.6 applies for members supporting expansion joints.

(3) 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 using the enhanced impact factors given in EN 1991‑2 for the area close to the expansion joints.

### 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 dampers, i.e. oscillation dampers.

NOTE Guidance on members supporting expansion joints is given in EN 1990:2023, A2.

## Performance criteria for pedestrian bridges

(1) For footbridges and cycle bridges where 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.

NOTE See EN 1990:2023, A.2, considering the comfort level defined in Annex H.

## 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 1 Guidance on the determination of fatigue loads from vortex excitation is given in EN 1991‑1‑4.

NOTE 2 For hangers in tied arch bridges, see Annex A.

## 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 EN ISO 12944‑3.

(2) All parts should, where practicable, 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 sections) 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 Clause 6.

## Drainage

(1) 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.

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

(3) For single or double-track 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, unless otherwise specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

# Fatigue

## General

### Requirements for fatigue verification

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

NOTE 1 EN 1993‑1‑9 provides two design concepts, which are the safe life concept and the damage tolerant concept. The National Annex can give the choice between these design concepts for the verification of the fatigue design situation.

NOTE 2 For orthotropic bridge decks, the following structural elements are critical:

1. for bridge decks with longitudinal stiffeners and crossbeams

* deck plate
* stiffeners
* crossbeams
* stiffener to crossbeam connections

1. for bridge decks with transverse stiffeners only

* deck plate
* stiffeners

NOTE 3 For fatigue verification of tension components, see EN 1993‑1‑11.

(2) Elements or parts of elements that cannot be inspected should be verified using the safe life concept according EN 1993‑1‑9.

(3) Fatigue verification is not required for pedestrian bridges, bridges carrying canals or other bridges that are predominantly statically loaded, unless such bridges or parts of them are likely to be dynamically excited by wind loads or pedestrians; parts of railway or road bridges that are neither subjected to actions expected to induce fatigue, nor likely to be excited by wind loads.

NOTE The National Annex can specify further structural elements where no fatigue verification is necessary.

### Design of road bridges for fatigue

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

(2) For fatigue verification of orthotropic steel decks, CEN/TS 1993‑1‑901 2 may be used.

NOTE The National Annex can specify conditions for the application of CEN/TS 1993‑1‑901 2.

(3) As an alternative to performing a fatigue verification, Annex C may be used for orthotropic steel decks.

NOTE The National Annex can specify conditions for the application of Annex C.

### Design of railway bridges for fatigue

(1) Fatigue verification should be carried out for all structural elements susceptible to fatigue.

(2) For critical regions for fatigue checks see Figure 10.1 and Figure 10.2 and Table 10.8.



Key

|  |  |
| --- | --- |
| 1 | at the connection between crossbeam and deck plate |
| 2 | at the connection between stiffener and deck plate |
| 3 | at the connection between crossbeam and stiffener |
| 4 | at a splice location |
| 5 | at cut-out location |

Figure 10.1 — Critical regions for fatigue, see also Table 10.8



Key

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

Figure 10.2 — Stiffeners with splice plates and metallic backing strips

(3) Structural detailing of steel bridge decks may be taken from Annex C.

NOTE The National Annex can specify conditions for the application of Annex C.

## Fatigue loading

### 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.

NOTE For hangers of tied arch bridges, see Annex A.

### Simplified fatigue load model for road bridges

(1) For the fatigue verification 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 The National Annex can give conditions for the applicability of fatigue load model 3 and specify alternative fatigue load model and the use of the procedure to obtain stress range spectra according to prEN 1993‑1‑9:2023, Annex A.

### Simplified fatigue load model for railway bridges

(1) For the fatigue verification of railway bridges the characteristic values for load model 71 should be used, excluding the load classification factor (*α*-factor) as given in EN 1991‑2.

NOTE The National Annex can give conditions for the applicability of load model 71 and specify alternative fatigue load models and the use of the procedure to obtain stress range spectra according to prEN 1993‑1‑9:2023, Annex A.

## Fatigue stress range

### General

(1) Unless a more refined method based on stress spectra is used, the following procedure using the simplified fatigue loading specified in 10.2.2 or 10.2.3 may be used to determine the design stress range.

(2) The maximum stress *σ*P,max and the minimum stress *σ*P,min should be determined by evaluating influence areas.

(3) The reference stress range Δ*σ*p for determining the damage effects of the stress range spectrum should be obtained from Formula (10.1).

|  |  |
| --- | --- |
|  | (10.1) |

(4) The damage effects of the stress range spectrum may be represented by the damage equivalent stress range related to 2 × 106 cycles, given by Formula (10.2):

|  |  |
| --- | --- |
|  | (10.2) |

where

|  |  |
| --- | --- |
| *λ* | is the damage equivalent factor as defined in 10.4; |
| *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.

### Analysis for fatigue

#### Longitudinal stiffeners

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

#### Crossbeams

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

(2) Where crossbeams are provided with cut-outs as given in Figure 10.3, and the crossbeams are designed with the nominal stress method, the action effects may be determined with a Vierendeel-model, where the deck plate and a part of the crossbeam below the cut-outs are the flanges and the areas between the cut-outs are the posts.



Key

|  |  |
| --- | --- |
| 1 | posts |
| 2 | flange for the upper chord, and flange and part of the web for the lower chord |

Figure 10.3 — Vierendeel-model for a crossbeam

(3) 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 Vierendeel-beam components due to bending moments, axial forces and shear forces to the overall deformation;
3. the effects of shear between the deck plate and the web of the crossbeam on the direct stresses and shear stresses at the critical section in Figure 10.4;
4. the effects of loads from the stiffeners into the web;
5. the shear stresses from the horizontal and vertical shear in the critical section in Figure 10.4. Information is provided in C.2.5.3.



Figure 10.4 — Stress distribution at cut-out

(4) The direct stresses in the critical section in Figure 10.4 may be determined using the Formulae (10.3) and (10.4):

|  |  |
| --- | --- |
|  | (10.3) |
|  | (10.4) |

where

|  |  |  |
| --- | --- | --- |
|  | are the stresses due to bending | (10.5) |
|  | are the compressive stresses due to local load from stiffeners | (10.6) |

|  |  |
| --- | --- |
| *V*Ed = *N*i+1 − *N*i | is the design value of the horizontal shear force; |
| *M*Ed = *V*Ed*h* | is the design value of the bending moment in the critical section; |
| *Fi*, *Fi*+1 | are the loads introduced from the stiffeners; |
| *t*w,cb | is the plate thickness of the crossbeam web. |

(5) Where no cut-outs are provided, the stresses at the critical section may be determined considering an I cross-section for the bottom chord comprising the crossbeam bottom flange, the depth of crossbeam web below the stiffener and a portion of the stiffener itself acting as a flange with an effective width *b*eff = 5*t*w,st, where *t*w,st is the plate thickness of the stiffeners.

## Fatigue verification procedures

### Fatigue verification

(1) The fatigue verification should be carried out using the criteria according to the criteria in Formulae (10.7) and (10.8):

|  |  |
| --- | --- |
|  | (10.7) |

and

|  |  |
| --- | --- |
|  | (10.8) |

where

|  |  |
| --- | --- |
|  | is the damage equivalent normal stress range related to 2 × 106 cycles; |
|  | is the characteristic reference value of fatigue resistance in terms of normal stress, at 2 × 106 cycles; |
|  | is the damage equivalent shear stress range related to 2 × 106 cycles; |
|  | is the characteristic reference value of fatigue resistance in terms of shear stress, at 2 × 106 cycles; |
|  | is the partial factor for the fatigue action effect, see 4.4(1); |
|  | is the partial factor for the fatigue resistance, see 4.4(2). |

NOTE For multiaxial fatigue verification, see EN 1993‑1‑9.

### Damage equivalent factors *λ* for road bridges

(1) The damage equivalent factor *λ* for road bridges up to 200 m span should be obtained from the Formula (10.9):

|  |  |
| --- | --- |
|  | (10.9) |

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 service 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. |

NOTE The values *λ*1, *λ*2, *λ*3, *λ*4 and *λ*max are determined in Annex F unless the National Annex gives other methods.

### Damage equivalent factors *λ* for railway bridges

(1) The damage equivalent factor *λ* for railway bridges with a span up to 100 m should be determined using Formula (10.10):

|  |  |
| --- | --- |
|  | (10.10) |

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 service 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 (8). |

(2) *λ*1 may be obtained from Table 10.3 and Table 10.4.

NOTE 1 The National Annex can give further conditions on the use of Table 10.3 or Table 10.4.

NOTE 2 The values given in Table 10.3 and Table 10.4 for mixed traffic correspond to the combination of train types given in Annex D of FprEN 1991‑2:2023.

NOTE 3 For lines with train type combinations other than those taken into consideration (specialised lines for example), the National Annex can specify values of *λ*1.

Table 10.3 — *λ*1 for standard rail traffic

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

Table 10.4 — *λ*1 for express multiple units and underground and for rail traffic with 25t 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 |

(3) In determining *λ*1, the critical length of the influence line should be taken as follows:

1. for moments:

* for a simply supported span, the span length, *L*i;
* for continuous spans in midspan sections, see Figure 10.7, the span length *L*i of the span under consideration;
* for continuous spans in support sections, see Figure 10.7, the mean of the two spans *L*i and *L*j adjacent to that support;
* for cross-girders supporting rail bearers (or stiffeners), the sum of the two adjacent spans of the rail-bearers (or stiffeners) 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.

NOTE The National Annex can give guidance on the critical length *L*i.

1. for shear for both a simply-supported span and a continuous span:

* for the support section, see Figure 10.7, the span under consideration *L*i;
* for the midspan section, see Figure 10.7, 0,4 × the span under consideration *L*i.



Key

|  |  |
| --- | --- |
| 1 | Midspan section |
| 2 | Support section |

Figure 10.7 — Location of midspan or support section

(4) *λ*2 should be obtained from Table 10.5.

Table 10.5 — *λ*2

|  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Traffic per year [106 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 |

(5) *λ*3 should be obtained from Table 10.6.

Table 10.6 — *λ*3

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| Design service 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 |

(6) λ4 should be obtained from Table 10.7.

Table 10.7 — *λ*4

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Δ*σ*1/Δ*σ*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 |
| Δ*σ*1 is the stress range at the section to be checked due to load model 71 on one track;  Δ*σ*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 10.7 is only valid if Δ*σ*1 and Δ*σ*1+2 have the same sign.

(7) The values of *λ*4 in Table 10.7 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 calculated using Formula (10.11):

|  |  |
| --- | --- |
|  | (10.11) |

where

|  |  |
| --- | --- |
| *n* | is the fraction of total traffic crossing the bridge whilst there is traffic on the other track. |

(8) The value of *λ* should not exceed *λ*max given by Formula (10.12):

|  |  |
| --- | --- |
|  | (10.12) |

### Combination of damage from local and global stress ranges

(1) For the simplified fatigue load models specified in 10.2.2 or 10.2.3, where the stress verification in a member is due to the combined effects of bending of the bridge (global effects) and bending of the internal elements (local effects), the combined effects Δ*σ*E,2 should be calculated using Formula (10.13):

|  |  |
| --- | --- |
|  | (10.13) |

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

## Fatigue resistance

(1) EN 1993‑1‑9 should be used for the fatigue resistance of bridges.

NOTE 1 The National Annex can exclude particular details in EN 1993‑1‑9 for the design of bridges.

NOTE 2 The National Annex can give supplementary requirements for the fatigue of orthotropic decks.

(2) For double fillet welds subject to out-of-plane bending, the detail category given in Table 10.8 should be used with the nominal stress method.

Table 10.8 — Detail category of double fillet welds subject to out-of-plane bending

| Detail category | Constructional detail | Symbol | Description | Supplementary requirements a b c |
| --- | --- | --- | --- | --- |
| 90 |  |  | Ein Bild, das Symbol, Kreis, weiß, Logo enthält.  Automatisch generierte Beschreibung Webs subject to normal stress by pure out-of-plane bending  — with weld toe failure (see b))  — with web angle 65° ≤ *α* ≤ 115° | Δ*σ* should be calculated using nominal normal stress in parent metal of the web. |
| Key  pure out-of-plane web bending  Z Zoom |
| a Size effect for *t* > 25 mm should be considered by stress modification with *k*s = (25/*t*)0,4 where *t* is the web plate thickness in mm.  b For partial penetration butt welds or fillet welds with weld sizes *a* > 0,4 *t*, only weld toe failure as assumed for  needs to be verified. For smaller weld sizes, weld root failure should also be considered according to Table 10.6 of prEN 1993‑1‑9:2023.  c The fatigue resistance curve is not applicable for a number of load cycles lower than 75 000. | | | | |

## Post weld treatment

(1) Where appropriate, weld improvement techniques such as weld toe grinding, TIG remelting of the weld toe region or mechanical impact treatment may be used to improve the fatigue life of connections.

NOTE 1 The National Annex can give provisions for post weld treatment.

NOTE 2 Design rules for High Frequency Mechanical Impact treatment (HFMI) are given in prEN 1993‑1‑9:2023, Annex F.

# Fasteners, welds, connections and joints

## Connections using bolts, rivets or pins

### General

(1) The rules in FprEN 1993‑1‑8:2023, Clause 5 apply taking into account the following provisions.

### Injection bolts

(1) The rules for injection bolts according to FprEN 1993‑1‑8:2023, 5.4.2 apply.

NOTE The National Annex can give restrictions on the use of injection bolts.

### Hybrid connections

(1) The rules for hybrid connections according to FprEN 1993‑1‑8:2023, 5.4.3 apply.

NOTE The National Annex can give restrictions on the use of hybrid connections.

### Connections with lug angles

(1) The eccentricity of the loads should be considered, particularly for fatigue verifications.

### Bolts on threaded holes

(1) The rules for bolts on threaded holes according to FprEN 1993-1-8:2023, 5.7.4 apply.

NOTE The National Annex can give restrictions on the use of bolts on threaded holes.

### Angles connected by one leg

(1) Single bolt connections should not be used for connecting structural members.

### 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.

## Welded connections

### General

(1) The rules in FprEN 1993-1-8:2023, Clause 6 apply taking into account the following provisions.

### Intermittent fillet welds

(1) The rules for intermittent fillet welds according to FprEN 1993-1-8:2023, 6.3.2.3 apply.

NOTE The National Annex can give restrictions on the use of intermittent fillet welds.

### Plug welds

(1) The rules for plug welds according to FprEN 1993-1-8:2023, 6.3.5 apply.

NOTE The National Annex can give restrictions on the use of plug welds.

### Flare groove welds

(1) The rules for flare groove welds according to FprEN 1993-1-8:2023, 6.3.6 apply.

NOTE The National Annex can give restrictions on the use of flare groove welds.

### Distribution of forces

(1) The rules in FprEN 1993-1-8:2023, 6.9 apply, except 6.9(5). Large plastic strains are not allowed on welded connections for getting any joint rotation on the welded connection.

(2) Only in case of accidental design situations, rules in 6.9(5) may be used.

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

(1) Single fillet or single-sided partial penetration butt welds where a bending moment is transmitted about the longitudinal axis of the weld producing tension on the root of the weld should be avoided where possible.

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

## Structural joints connecting H- or I-sections

(1) For this type of joint, 7.1.2(2) should be considered.

NOTE The rules in FprEN 1993-1-8:2023, Clause 8 usually do not apply for typical bridge structures. The National Annex can give further conditions on the use of structural joints connecting H- and I-sections.

## Hollow section joints

(1) The rules in FprEN 1993-1-8:2023, Clause 9 apply considering the field of application.

NOTE The National Annex can give conditions on the use of structural joints connecting hollow sections.

(2) For fatigue verifications, secondary moments and eccentricities at the joints should be taken into account. See EN 1993‑1‑9.

1. (normative)  
     
   Design of hangers for tied-arch bridges
   1. Use of this Annex

(1) This Normative Annex contains additional provisions to 5.4 for the design of:

* welded and forged round bar steel hangers for tied-arch bridges;
* welded flat steel plate hangers for tied-arch bridges and,
* rope hangers for tied-arch bridges,

subjected to cyclic stresses due to traffic and to wind-induced or rain-wind-induced oscillations.

* 1. Scope and field of application

(1) This Normative Annex applies to hangers for tied-arch bridges.

(2) Design rules and recommendations for the appropriate design for fatigue and to prevent aero-elastic instability are given.

NOTE Further background information, guidance on dynamic calculation methods and relevant calculation examples are given in the guideline “Tension members in bridge construction susceptible to vibration — Design rules for hangers of tied-arch bridges and recommendations for design for fatigue”.

(3) Cross-wind vortex-induced vibrations arise when the air flowing around a hanger causes regular vortex shedding at the natural frequency of the hanger and the resulting forces give rise to resonances.

(4) Rain-wind-induced oscillations are triggered when vibrations in the hangers and wind action cause water trickling down hangers with a circular cross-section to oscillate around the hanger profile, giving rise to rhythmically fluctuating variable distributions of pressure with the corresponding force actions.

(5) Galloping vibrations are caused by cycling asymmetrical distributions of wind pressure on the cross-section accompanying the proper motion of the hangers. They may give rise to aeroelastic instability. Flat steel plate hangers and circular sections with ice are affected in particular.

(6) Traffic-induced fatigue actions on hangers are essentially caused by cycling axial forces and bending moments at the ends of the hangers caused by restraint which give rise to stresses in the joints in particular.

NOTE The rules were developed with reference to the simplified fatigue load model.

(7) The stresses due to wind and traffic should generally be considered jointly when designing for fatigue.

* 1. Design principles
     1. Material and cross-sections for tension members

(1) Hangers and joints shall be designed for fatigue and executed accordingly.

(2) Where it is not possible to avoid joints in round bar steel hangers, full penetration welding should be used and a welding procedure test should be performed for the weld joint according to EN 1090‑2.

In case of other types of joint, fatigue verification of the joint should be carried out.

(3) Hangers should be made of high-strength ductile materials and with the highest possible degree of slenderness in order to reduce restraint forces and to enhance robustness.

(4) In the absence of specific calculations or testing to determine toughness requirements, recommended values given in Table A.1 for the maximum diameter of round bar steel hangers may be used for steel grades and qualities according to EN 10025(all parts).

Table A.1 — Recommended values for the maximum diameter of round bar steel hangers

|  |  |  |  |
| --- | --- | --- | --- |
| S355 J2 | S355 K2/S355 N | S355 NL | S460 NL |
| 100 mm | 130 mm | 160 mm | 160 mm |

* + 1. Design recommendations for welded connections of round bar steel hangers

(1) The following procedure for the design of welded hanger connections according to the geometry of Figure A.1 is recommended for the purpose of providing adequate fatigue resistance:

|  |  |  |
| --- | --- | --- |
| 1. Diameter of hanger: |  | (A.1) |

where

|  |  |
| --- | --- |
|  | is the design value of the axial force for the persistent design situation according to EN 1990; |
|  | is the design tensile strength of the hanger according to Table A.2. |

|  |  |  |
| --- | --- | --- |
| 1. Thickness of gusset plate: |  | (A.2) |
| 1. Width at cope hole level: |  | (A.3) |

where

|  |  |
| --- | --- |
|  | is the design tensile strength at cope hole level according to Table A.2. |

|  |  |  |
| --- | --- | --- |
| 1. Embedded length: |  | (A.4) |

where

|  |  |
| --- | --- |
|  | is the design shear strength at cope hole level according to Table A.2. |

|  |  |  |
| --- | --- | --- |
| 1. Maximum width of plate: |  | (A.5) |
| 1. Outside radius: |  | (A.6) |
| 1. Clear height of gusset plate: |  | (A.7) |

1. *EI*i is the averaged bending stiffness of the segment i according to Figure A.1.

NOTE Depending on the availability of the products, the final dimensions can be chosen in such a way that the Formulae (A.1) to (A.7) are satisfied.



Key

|  |  |
| --- | --- |
| 1 | Hole of diameter *D* |
| 2 | Weld (full penetration) |
| 3 | Simplified modelling |

Figure A.1 — Geometry of the hanger connection plates and the hanger diameter in welded end connections

Table A.2 — Design strength in N/mm2

| Steel grade | *σ*Rd | *σ*net,Rd | *τ*Rd |
| --- | --- | --- | --- |
| S355 | 190 | 175 | 60 |
| S460 | 240 | 225 | 80 |

* + 1. Design recommendations for forged hangers

(1) The following procedure for the design of forged hanger connections according to the geometry of Figure A.2 is recommended for the purpose of providing adequate fatigue resistance:

|  |  |  |
| --- | --- | --- |
| 1. Diameter of hanger: |  | (A.8) |

where

|  |  |
| --- | --- |
|  | is the design value of the axial force for the persistent design situation according to EN 1990; |
|  | is the design tensile strength of the hanger according to Table A.2. |

|  |  |  |
| --- | --- | --- |
| 1. Thickness of gusset plate: |  | (A.9) |
| 1. Width of weld (full penetration): |  | (A.10) |
| 1. Length of forged hanger end: |  | (A.11) |
| 1. Maximum width of plate: |  | (A.12) |
| 1. Clear height of gusset plate: |  | (A.13) |



Key

|  |  |
| --- | --- |
| 1 | Weld (full penetration) |

Figure A.2 — Geometry of the hanger connection plates and the hanger diameter in forged end connections

(2) The scope of application of forged hangers is limited to diameters *D* between 70 mm and 170 mm. The largest possible outside radius *r* is to be selected.

(3) In design verifications, imperfections according to Figure A.3 should be accounted for.

|  |  |  |
| --- | --- | --- |
| Eccentricity | | Torsion |
|  |  | Interrelated torsion of forged hanger:  Max *φ*s = *φ*0 + *φ*u = *t*/*b*\*  Torsion between forged hanger end and connected plate:  Max *φ*0 = Max *φ*u = *t*/(2*b*\*)  Or *v* ≤ *t*/4 |
| *φ*0 or *φ*u |
|  |
|  |  |  |

Key

|  |  |
| --- | --- |
| 1 | As built |
| 2 | As designed |
| 3 | Axis round steel |
| 4 | Axis gusset plate |

Figure A.3 — Imperfections for the design of forged end connections

(4) Proof that the required material properties have been achieved should be provided by means of testing identical samples produced at the same time as the workpiece.

* + 1. Design recommendations for flat steel plate hangers

(1) To minimize the risk of aero-elastic effects, it is recommended that the dimensions of flat steel plate hangers fulfil the conditions in Formula (A.14):

|  |  |
| --- | --- |
|  | (A.14) |

where

|  |  |
| --- | --- |
| *b* | is the width of the plate; |
| *d* | is the thickness of the plate. |

* + 1. Design recommendations for rope hangers

(1) The following procedure for the design of rope hangers with pinned connections according to the geometry of Figure A.4 is recommended for the purpose of providing adequate fatigue resistance:

1. Thickness of gusset plate: *t* = 0,4 *ϕ*R
2. Maximum width of plate:
3. Clear height of gusset plate: *L*F ≥ *ϕ*A



Key

|  |  |
| --- | --- |
| 1 | Threaded bar |
| 2 | Rope exit |
| 3 | Free rope length |

Figure A.4 — Geometry of the hanger connection plates of rope hangers with pinned connection

(2) The rope diameter ØR should be determined according to A.6. Specifications on how to determine *b*F are given in A.3.2. The values *Ø*A, *Ø*B, *t*B and *Ø*G as well as assumptions on the anchoring elements mean dimensions *Ø*H-i are to be found in or derived from the manufacturer’s specifications.

* + 1. Measures to reduce restraint forces from the main structure

(1) Restraint forces due to deformations of the main structure may be reduced by appropriate orientation of the gusset plates (flexible connection at right angles to the plane of the plate).

NOTE The gusset plates at the stiffening girder and at the arch can be placed at right angles to each other to reduce restraint forces due to deformations transverse to the plane of the arch.

(2) Stiffening girders and crossbeams should have sufficient flexural stiffness to reduce the restraint forces due to traffic-induced deformations of the structure.

NOTE Increasing the torsional stiffness of the stiffening girder (box girder) can be beneficial.

* 1. Design rules for round bar steel hangers
     1. Application limits

(1) For tied-arch bridges with welded or forged round bar steel hangers and spans equal to or less than 60 m, fatigue verification of the hanger connections to take account of wind-induced oscillations may be omitted if the design recommendations given in A.3.2, A.3.3 and A.3.6 are followed.

* + 1. Oscillations due to vortex shedding

(1) In the case of round bar steel hangers, the assessment for oscillations due to vortex shedding should be carried out for the fundamental vibrations and all higher vibration modes with frequencies *f*i lower than 10 Hz. The calculations should be performed in both directions (in the plane of the arch and at right angles to the plane of the arch).

(2) If no detailed non-linear dynamic analysis is done, the verification may be based on the lateral load given by Formula (A.15):

|  |  |
| --- | --- |
|  | (A.15) |

where

|  |  |  |  |
| --- | --- | --- | --- |
|  | is the critical wind velocity of the relevant mode, calculated from Formula (A.16): | | |
|  | | | (A.16) |
|  | is the diameter of hangers in m | | |
|  | is the natural frequency in Hz of the relevant mode taking account the permanent axial force acting on the hangers | | |
|  | is the Strouhal number ( = 0,20 for circular cylinders) | | |
|  | is a factor to take account of a continuous reduction in the level of excitation as the natural frequencies increase: | | |
|  | | for *f*i < 7 Hz | (A.17) |
|  | | for 7 Hz ≤ *f*i < 10 Hz | (A.18) |

(3) The lateral load *q* should be assumed to be a constant action acting on an effective correlation length *L*W,i of 24 *D*, at the maximum oscillations of the individual modes (Figure A.5).

NOTE For oscillations due to vortex shedding, *L*w,1 = *L*w,2 in Figure A.5.



Figure A.5 — Schemes of load application

(4) The bending moments should be calculated using second order analysis, taking account of the characteristic value of the tensile force of the hangers due to self-weight and the geometry of the connection to the tension member (graduated stiffnesses in accordance with Figure A.1).

(5) If a logarithmic decrement of damping *δ*meas greater than 0,001 5 is obtained by measurements of the structure, the lateral load *q* may be linearly reduced according to Formula (A.19):

|  |  |
| --- | --- |
|  | (A.19) |

NOTE In certain cases (e.g. if higher level of damping is anticipated), it can be useful to check the level of damping by carrying out measurements of the finished structure, i.e. after the surfacing layer has been laid and after installation of the parapets.

(6) The stress range at the points likely to be affected by fatigue should be calculated from Formula (A.20):

|  |  |
| --- | --- |
|  | (A.20) |

where

|  |  |
| --- | --- |
| *M*w,max | is the calculated bending moment |
| *W*el | is the elastic section modulus |

NOTE The stress range obtained by Formula (A.20) is the stress range equivalent to the damage caused by two million load cycles, where the damage equivalence is taken into account by increasing the effective correlation length in accordance with (3).

* + 1. Rain-wind-induced oscillations

(1) Rain-wind-induced oscillations should be verified for tension members with a round bar if the diameter of the hangers exceeds 65 mm and the fundamental frequency *f* is less than 6,5 Hz.

(2) If not assessed by a more detailed calculation, the verification may be based on the lateral load calculated with the Formula (A.21):

|  |  |
| --- | --- |
| in kN/m but | (A.21) |

where

|  |  |  |  |
| --- | --- | --- | --- |
| *v*crit,i | is the critical wind velocity for each mode, calculated with the Formula (A.22): | | |
|  | | | (A.22) |
| *f*0 | is the reference frequency, *f*0 = 1 Hz | | |
|  | is the factor to take account of the continuous decrease in the excitation at wind velocities in excess of 20 m/s: | | |
|  | | for *v*crit,i ≤ 20 m/s | (A.23a) |
|  | | for 20 m/s < *v*crit,i ≤ 3 m/s | (A.23b) |
|  | is the factor to take account of the continuous reduction of the excitation risk due to rain-wind-induced vibrations with a decreasing diameter: | | |
|  | | for *D* ≤ 0,065 m | (A.24a) |
|  | | for 0,065 m < *D* < 0,072 m | (A.24b) |
|  | | for *D* ≥ 0,072 m | (A.24c) |
| *c* | is the exciting coefficient, which is dependent on the angle of inclination *α* of the hanger, irrespective of the wind direction and the hanger oscillation direction under consideration, see Figure A.6. Irrespective of the angle of inclination, the exciting coefficient should be set at 0,04 at least. | | |



Figure A.6 — Exciting coefficient *c*

|  |  |  |
| --- | --- | --- |
| *q*max | is the maximum applicable lateral load calculated with Formula (A.25): | |
|  | | (A.25) |
| *q*0 | is the reference value of the lateral load, *q*0 = 1 kN/m | |
| *D*0 | is the reference value of the hanger diameter, *D*0 = 1 m | |

(3) If a logarithmic decrement of damping *δ*meas greater than 0,0 015 is obtained by measurements of the structure, the lateral load *q* may be linearly reduced in accordance with Formula (A.19).

(4) The lateral load *q* may be considered as a constant action on an effective correlation length *L*w,i = 0,27 *L*i in accordance with Figure A.5. The calculation follows A.4.2. The maximum stress amplitude due to rain-wind-induced vibration is calculated with the Formula (A.26):

|  |  |
| --- | --- |
|  | (A.26) |

where

|  |  |
| --- | --- |
| *M*rw,max | is the calculated bending moment; |
| *W*el | is the elastic section modulus. |

* + 1. Traffic-induced stresses

(1) The second order effects should be taken into account when determining the fatigue stress ranges due to restraint forces in the hangers caused by deformation.

* + 1. Verification concepts
       1. Verification concept for traffic and oscillations due to vortex shedding

(1) A fatigue verification using the fatigue resistance Δ*σ*c should be performed as specified in Clause 10.

(2) For each mode, the stress ranges due to traffic and oscillations due to vortex shedding should be added according to the criteria in Formula (A.27):

|  |  |
| --- | --- |
|  | (A.27) |

where

|  |  |
| --- | --- |
| Δ*σ*e,2 | is the fatigue stress range due to traffic in accordance with Clause 10; |
| Δ*σ*w,e,2 | is the stress range obtained by (A.20); |
| Δ*σ*c | is the fatigue resistance of the detail category under consideration; |
| *γ*Mf | is the partial factor for the fatigue resistance of the main structural members. |

(3) Alternatively, a fatigue resistance assessment based on long-term measurements may be performed.

* + - 1. Verification concept for rain-wind-induced vibrations
         1. Ultimate limit state verification

(1) An ultimate limit state verification for accidental design situations should be performed for rain-wind-induced vibrations by applying the criterion in Formula (A.28):

|  |  |
| --- | --- |
|  | (A.28) |

where

|  |  |
| --- | --- |
| *σ*G | is the stress due to the axial force acting on the hangers under permanent actions; |
| *σ*Q | is the stress due to the axial force and bending moment of the hanger as a result of frequent traffic loads; |
| *σ*rw | is the stress due to the bending moment of the hanger as a result of rain-wind-induced vibrations obtained by the Formula (A.26); |
| *f*y | is the yield strength of steel. |

* + - * 1. Fatigue verification

(1) The fatigue verification should be performed according to the criterion in Formula (A.29):

|  |  |
| --- | --- |
|  | (A.29) |

where

|  |  |  |
| --- | --- | --- |
| *k*H,j | is the reduction factor given by the Formula (A.30), used to determine the equivalent stress range, taking account of the frequency with which rain-wind-induced vibrations occur: | |
|  | | (A.30) |
| *v*0 | is the reference value of the critical wind velocity: *v*0 = 1 m/s; | |
| *v*crit,i | is the critical wind velocity, in m/s, obtained by the Formula (A.22); | |
|  | is the stress range due to rain-wind-induced vibrations: | |
|  | in accordance with the Formula (A.26) | |

* 1. Design of flat steel plate hangers
     1. Oscillations due to vortex shedding

(1) Oscillations due to vortex shedding should be verified for all modes of flat steel plate hangers with natural frequencies *f*i less than 10 Hz.



Key

|  |  |
| --- | --- |
| 1 | Wind direction |

Figure A.7 — Dimension ratios for oscillations due to vortex shedding

(2) The lateral load should be calculated with the Formula (A.31):

|  |  |
| --- | --- |
| in kN/m | (A.31) |

where

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *v*crit,i | is the critical wind velocity, in m/s, of the relevant mode, given by the Formula (A.32); | | | |
|  | | | | (A.32) |
| *b* | is the height of the cross-section as shown in Figure A.7, in m; | | | |
| *c*lat | is the exciting power coefficient for rectangular cross-sections; | | | |
| *c*lat = 1,1 | | for 0 ≤ *b*/*d* ≤ 4 | | (A.33) |
|  | | for 4 < *b*/*d* ≤ 8 | | (A.34) |
| *c*lat = 0,7 | | for 8 < *b*/*d* | | (A.35) |
| *k*F,i | is the reduction factor according to the Formulae (A.17) and (A.18); | | | |
| *k*T,i | is the reduction factor to take account of turbulence; | | | |
| *k*T,i = 1,0 | | for *v*cr,i ≤ 8 m/s | | (A.36) |
|  | | for *v*cr,i > 8 m/s | | (A.37) |
| *k*H,i | is the reduction factor to take account of frequency; | | | |
|  | | | but *k*H,i ≤ 1,0 | (A.38) |
| *St* | is the Strouhal number for rectangular cross-sections according to Table A.3. | | | |

Table A.3 — Strouhal number

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| *b*/*d* | 0,1 | 0,2 | 0,285 | 0,34 to 0,5 | 1 to 8 |
| Strouhal number *St* | 0,09 | 0,11 | 0,15 | 0,06 | 0,12 |
| Intermediate values may be obtained by linear interpolation. | | | | | |

(3) The bending moments (for each mode) should be determined and the decrement of damping taken into account according to A.4.2.

(4) The lateral load *q* should be taken as a constant action on an effective correlation length *L*w,i = 24 *D* in accordance with Figure A.5. The stress range should be calculated according to the Formula (A.20).

(5) An assessment of the oscillations due to vortex-shedding around the weak axis is not required if the geometrical recommendations given in A.3.4 are followed and the hanger gusset plates are designed with favourable detail categories.

* + 1. Galloping
       1. Onset wind velocities for galloping oscillations in the bending mode

(1) Rectangular hangers with an aspect ratio 1,0 ≤ *b*/*d* ≤ 3,0 should be checked for galloping oscillations in the bending mode. The relevant onset wind velocity is caused by wind attack on the narrow side *d* of the hanger as shown in Figure A.8.



Key

|  |  |
| --- | --- |
| 1 | Wind direction |

Figure A.8 — Ratio of dimensions for galloping oscillations in the bending mode

(2) The onset wind velocity is determined with Formula (A.39):

|  |  |
| --- | --- |
|  | (A.39) |

where

|  |  |
| --- | --- |
| *m*L | is the mass per unit length in kg/m; |
| *f* | is the natural frequency, in Hz, of the first bending mode perpendicular to the wind direction (i.e. around the weak axis), taking account of the characteristic value of the axial force acting on the hangers due to self-weight; |
| *δ* | is the logarithmic decrement of damping of the relevant mode: δ = 0,001 5; |
| *d* | is the height of the section perpendicular to the wind direction in accordance with Figure A.8, in m; |
| *ρ* | is the density of the air in kg/m3: *ρ* = 1,25 kg/m3; |
| *a*0, *b*0, *c*0 | are stability values for galloping oscillations in the bending mode according to Table A.4. |

Table A.4 — Stability values for galloping oscillations in the bending mode

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *b*/*d* | 1,0 | 1,5 | 2,0 | 3,0 |
| *a*0 | −8,0 | 20 | 40 | 270 |
| *b*0 | 0,6 | 2,0 | 4,0 | 5,0 |
| *c*0 | 10 | 20 | 25 | 55 |
| Intermediate values may be obtained by linear interpolation. | | | | |

* + - 1. Onset wind velocities for galloping oscillations in the torsional mode

(1) Rectangular hangers with an aspect ratio *b*/*d* equal to or greater than 3,0 should be checked for galloping oscillations in the torsional mode.

(2) The onset wind velocity should be determined with Formula (A.40):

|  |  |
| --- | --- |
|  | (A.40) |

where

|  |  |  |
| --- | --- | --- |
| *Θ* | is the mass moment of inertia in kg m2/m: | |
|  | | (A.41) |
| *δ*T | is the logarithmic decrement of damping of the first torsional mode, with *δ*T = 0,001 5 | |
| *d, b* | are the height and the width of the section in accordance with Figure A.8, in m; | |
| *f* | is the natural frequency, in Hz, of the first oscillation in the torsional mode where *δ*T = 0,001 5; | |
| *a*0, *b*0, *c*0 | are stability values for galloping oscillations in the torsional mode according to Table A.5. | |

Table A.5 — Stability values for galloping oscillations in the torsional mode

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *b*/*d* | 3,0 | 4,0 | 6,0 | 8,0 |
| *a*0 | 500 | 1 500 | 2 500 | 5 000 |
| *b*0 | 50 | 100 | 150 | 200 |
| *c*0 | 10 | 15 | 15 | 25 |
| Intermediate values may be obtained by linear interpolation. | | | | |

* + 1. Traffic-induced stresses

(1) The fatigue actions should be taken into account as specified in A.4.4.

* + 1. Verification concept

(1) The fatigue verification for the stress ranges due to traffic and oscillations due to vortex-shedding should be based on A.4.5.1.

(2) It should also be verified that the onset wind velocities for galloping oscillations in the bending and torsional modes exceed 1,25 times the mean velocity *v*m at the mid-height of the hangers in accordance with EN 1991‑1‑4:

|  |  |  |
| --- | --- | --- |
|  | for galloping oscillations in the bending mode and | (A.42) |
|  | for galloping oscillations in the torsional mode | (A.43) |

* 1. Design rules for rope hangers

(1) Rope hangers are to be made of full locked ropes according to EN 1993‑1‑11. The scope of application is limited to rope diameters *Ø*R between 20 mm and 55 mm.

(2) In the structural design of the rope, the planned exchange of a hanger should be accounted for as a transient design situation. The structural safety and the fatigue safety of the ropes should be verified according to EN 1993‑1‑11.

(3) The connection areas should be constructed according to the recommendations in Figure A.4. Sufficient fatigue safety of the gusset plates should be verified under the assumption that the bolt connection is rigid in both directions.

(4) The analysis of fatigue relevant bending in rope hangers caused by traffic load, by cross-wind vortex induced vibrations and rain-wind-induced vibrations may be omitted if the span length of the tied-arch bridge is no longer than 90 m and if the hangers connect the arches with the stiffening girders in regular spaced intervals.

The installation of length compensating elements is recommended. The fatigue safety of the integrated thread is to be verified.

1. (normative)  
     
   Supplementary rules for the design of plate girders curved in plan with rigid restraints to the compression flange
   1. Use of this annex

(1) This Normative Annex contains additional provisions to Clause 8 for the design of plate girders curved in plan.

* 1. Scope and field of application

(1) This Normative Annex covers plate girders curved in plan when the following conditions apply:

* The compression flange is braced by restraints which are rigid in accordance with 8.3.5(5) and the tension flange has continuous lateral restraint provided by a deck slab or a system of plan bracing.
* The web is stiffened by rigid transverse stiffeners.
* There are no longitudinal stiffeners on web or flanges.
* There are no concentrated torques applied to the beams between the rigid restraints, e.g. during construction before the deck slab is present.
* The flange curvature parameter *z*f (Formula (B.1)) is less than or equal to 9.
* The web curvature parameter *z*a (Formula (B.3)) is less than or equal to 50.
  1. Bending resistance

(1) For plate girders with uniform curvature in plan, the compression flange curvature parameter should be calculated defined according to Formula (B.1):

|  |  |
| --- | --- |
|  | (B.1) |

where

|  |  |
| --- | --- |
| *L*R | is the spacing of the centres of rigid lateral restraints to the compression flange; see Figure B.1; |
| *R*B | is the radius of the beam web axis in plan; see Figure B.1; |
| *b*c | is the width of compression flange. |



Key

|  |  |
| --- | --- |
| 1 | Rigid lateral restraint to compression flange defining the length *L*R |
| 2 | Continuous lateral restraint to top flange |

Figure B.1 — Definition of curved plate girder dimensions

Where *z*f ≤ 0,2, the beam may be treated as straight for the purposes of determining lateral torsional buckling resistance.

Where 0,2 < *z*f ≤ 9,0, the design buckling resistance should be determined in accordance with EN 1993‑1‑1:2022, 8.3.2, but with calculated according to Formula (B.2):

|  |  |
| --- | --- |
|  | (B.2) |

where *C*zf has the following values:

*C*zf = 0,2 for

*C*zf = 0,25 for

(4) When determining the relative slenderness in accordance with 8.3.2 of EN 1993‑1‑1:2022, the elastic critical moment, *M*cr, should be calculated for either the curved girder or an equivalent straight girder with the same restraints and restraint spacings.

* 1. Shear resistance

(1) For beams with curvature in plan, the web curvature parameter *z*a should be calculated according Formula (B.3):

|  |  |
| --- | --- |
|  | (B.3) |

where

|  |  |
| --- | --- |
| *R*B | is the radius of the beam web axis in plan; |
| *a*R | is the spacing of the rigid transverse stiffeners measured along the beam web axis; see Figure B.1; |
| *t*w | is the thickness of the web. |

(2) Where *z*a ≤ 1,0, the design shear buckling resistance may be determined in accordance with 7.2(1) of FprEN 1993-1-5:2023.

(3) Where 1,0 < *z*a ≤ 50, the design shear buckling resistance should be calculated according to Formula (B.4):

|  |  |
| --- | --- |
|  | (B.4) |

where

|  |  |  |  |
| --- | --- | --- | --- |
| *h*w | is the height of the web | | |
| *χ*w | is determined as follows: | | |
|  | | for |  |
|  | | for |  |
|  | | for |  |
| for | | | |
| for | | | |
|  | is the relative slenderness calculated according to (4) | | |
| *A*χ, *B*χ and *C*χ should be taken from Table B.1. | | | |

NOTE The above resistance becomes increasingly conservative for *a*/*hw* > 2,5.

Table B.1 — *A*χ, *B*χ and *C*χ parameters

| Relative slenderness |  |  |  |
| --- | --- | --- | --- |
|  |  |  |  |
|  |  |  | — |

(4) The relative slenderness should be calculated according to Formula (B.5):

|  |  |
| --- | --- |
|  | (B.5) |

where

and are defined in Table B.2.

Table B.2 — *A*k and *B*k parameters

| Value of |  |  |
| --- | --- | --- |
|  |  |  |
|  |  |  |

* 1. Interaction between shear force and bending moment

(1) The interaction between shear force and bending moment should be verified in accordance with 9.1(1) of FprEN 1993-1-5:2023 with the design shear resistance calculated in accordance with B.3 (2) or (3) as appropriate.

* 1. Design of restraints to the compression flange

(1) Restraints to curved in plan compression flanges shall be designed for the forces developed in resisting the lateral buckling deflections of the flange, considering both the initial intended curvature and geometrical imperfections. The restraint forces should be determined by second order analysis or by simplified methods which make allowance for second order effects.

(2) Restraints to curved in plan compression flanges may be designed in accordance with the principles of 8.3.5(5) provided that the additional radial force induced by the plan curvature is included. The force to apply to each discrete rigid compression flange restraint, *F*Ed, may conservatively be calculated by introducing the additional compression flange bow from intended plan curvature using Formula (B.6):

|  |  |
| --- | --- |
|  | (B.6) |

Where *N*Ed is defined in 8.3.5(1).

1. (informative)  
     
   Recommendations for the structural detailing of steel bridge decks
   1. Use of this annex

(1) This Informative Annex provides complementary and supplementary guidance to Clause 10 for the structural detailing of steel bridge decks.

NOTE National choice on the application of this Informative Annex can be given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This Informative Annex applies to road bridges and railway bridges.

* 1. Road bridges
     1. General

(1) This annex gives recommendations for the structural detailing, weld preparation, execution of steel bridge decks to enable the following to be achieved:

1. a minimum quality standard as specified in Clause 4;
2. a standard design with details for the deck plate and trough stiffeners for which no further fatigue verification is required.

The recommendations are based on a standard design as given in Figure C.1.

NOTE 1 The National Annex can give additional technical information on the structural detailing, weld preparation, execution of steel bridge decks.

NOTE 2 No recommendations are given for decks provided with transverse stiffeners.





Key

|  |  |
| --- | --- |
| 1 | heavy traffic lane |
| 2 | deck plate |
| 3 | welded connection of stiffener to deck plate |
| 4 | welded connection of stiffener to web of crossbeam |
| 5 | cut-out in web of crossbeam |
| 6 | splice of stiffener |
| 7 | splice of crossbeam |
| 8 | welded connection of crossbeam to main girder or transverse frame |
| 9 | welded connection of the web of crossbeam to the deck plate |

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

(3) The design recommendations mainly refer to the carriageway, see Figure C.1:

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 Table C.3, Table C.4 and Table C.5.

* + 1. Deck plate
       1. General

(1) Fatigue actions on the deck plate 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 (composite actions).

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



Key

|  |  |
| --- | --- |
| 1 | wheel load |

Figure C.2 — Bending of deck plate: 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 |

Key

|  |  |
| --- | --- |
| 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 welded connections between the deck plate and webs of main girders, webs of open section stiffeners and webs of crossbeams.

(6) The welded connection between the deck plate and the webs of the stiffeners is covered in C.3.3.

(7) In order to meet the required assembly tolerances, tolerances for the supplied deck plate should be specified. Reference values for the tolerances are given in Table C.3, line 1.

* + - 1. Thickness of deck plates and minimum stiffness of stiffeners

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

Recommended plate dimensions are as follows, see Figure C.2:

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

*t* ≥ 14 mm for asphalt layer ≥ 70 mm,

*t* ≥ 16 mm for asphalt layer ≥ 40 mm and < 70 mm,

*t* ≥ 20 mm for asphalt or epoxy layers < 40 mm.

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

max{*e*/*t* ; *a*/*t*} ≤ 25, recommended *e* ≤ 300 mm, recommended *a* ≤ 300 mm

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

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

*t* ≥ 10 mm and *e*/*t* ≤ 40

*e* ≤ 600 mm.

* 1. Thickness of stiffener:

*t*stiff ≥ 6 mm

NOTE 1 The National Annex can give guidance on the plate thickness to be used.

NOTE 2 If the conditions given in NOTE 1 are satisfied, the local bending moments in the deck plate can be neglected.

(2) For the minimum longitudinal stiffness of stiffeners, see Figure C.4. The minimum thickness of stiffeners amounts to 6 mm.

The minimum stiffness values in Figure C.4 are recommended.

NOTE The National Annex can give guidance on the minimum stiffness of stiffeners or can give different values.



Condition for curve A



Key

|  |  |
| --- | --- |
| X | Distance between cross girders *a* [m] |
| Y | Second moment of area *I*B of the stiffeners including deck plate [cm4] |
| 1 | traffic lane for heavy vehicles |
| 2 | web of main girder or longitudinal girder |

NOTE 1 Curve A applies to all longitudinal stiffeners that are not covered by b).

NOTE 2 Curve B applies to longitudinal stiffeners that are located under the most heavily loaded traffic lane within 1,20 m of a web of a main girder.

NOTE 3 The figure applies to all types of stiffeners.

Figure C.4 — Minimum stiffness of longitudinal stiffeners

(3) For deck plates for pedestrian bridges, non-trafficked central reserves and pavements, the dimensions should fulfil the following conditions:

*t* ≥ 10 mm and *e*/*t* ≥ 40

*e* ≤ 600 mm

NOTE Bending moments in deck plates do not require verification if recommendations (1), (2) and (3) are followed.

* + - 1. 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 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 backing strip



Figure C.6 — Deck plate welds transverse to traffic lane with backing strip (not recommended)

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

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



Key

|  |  |
| --- | --- |
| 1 | no sealing weld |

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

(4) V-welds with steel backing strips may be used for longitudinal welds provided the following requirements are satisfied:

1. execution in accordance with Figure C.7
2. tolerances and inspections given in Table C.4, line 2.
   * + 1. 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.



Key

|  |  |
| --- | --- |
| 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 closed section stiffeners to the deck plate, see C.3.3.

* + 1. Longitudinal stiffeners
       1. General

(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 due to shear forces in the stiffeners;
3. longitudinal stresses due to local bending moments and global stresses in the stiffeners as part of the main girder;
4. local bending moments at the connections between the stiffeners and the web of the crossbeams.
   * + 1. Type of stiffeners

(1) For closed stiffener forms, such as trapezoidal cross-section, V-shaped or semi-circular cross-section, Table C.3, line 2, should be used.

(2) For open section stiffeners, Table C.3, line 3, should be used.

(3) In the case of a change in the plate thickness of stiffeners, the misalignment at the surface of plates should not exceed 2 mm.

* + - 1. Stiffener to deck plate connection

(1) For closed section stiffeners, the weld between the stiffener and the deck plate should be a butt weld.

(2) Irrespective of the results of the structural analysis, the throat thickness should not be less than the wall thickness of the stiffener, see Table C.4, lines 3 and 4.

(3) For stiffener to deck plate connections outside the carriageway, see Table C.4, line 5.

(4) For tolerances and tests, see Table C.4, lines 3, 4 and 5.

* + - 1. Stiffener to stiffener connection

(1) The stiffener to stiffener connection should have splice plates in accordance with Table C.4, line 6.

(2) The splice should, as far as possible, be located at a distance of 0,2 *d*c from the crossbeam, where *d*cis the distance between adjacent crossbeams.

(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, line 6 is as follows:

1. first, the weld between the stiffener and the splice plate;
2. Secondly, the weld between the other stiffener and the splice plate; at position no. 1 and 2 shown in Table C.4, line 6, the bottom flange (position no. 1) should be welded first, followed by the web (position no. 2);
3. Finally, the deck plate weld.

(4) For the butt welds between the stiffener and the splice plate, the tolerances and inspections given in Table C.4, line 7 should apply.

* + - 1. Connection of stiffeners to the web of the crossbeam
         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 cut-outs and deformations on the stiffeners.

|  |  |
| --- | --- |
|  |  |
| Rotation of the stiffener at its connection to web of crossbeam, see C.3.3.5.1(1), b) | Imposed deformations to stiffener from strain distribution in the web of the crossbeam, see C.3.3.5.1(1), c) |

Figure C.9 — Connection of stiffeners to the web of the crossbeam

(2) The magnitude of these effects depends on whether:

1. the stiffeners pass through the web and the shapes of the cut-out and cope hole;
2. the stiffeners are fitted between the webs of the crossbeams.

(3) Stiffeners should preferably pass through cut-outs of the crossbeam.

(4) The method of construction in which the stiffeners do not pass through the webs (i.e. are fitted between the crossbeams) is only permitted in exceptional cases, e.g. for bridges with extremely small depths of crossbeams, small spacing of crossbeams or with light traffic only, see the provisions in C.3.3.5.3.

(5) For flat stiffeners, see Figure C.10, the fatigue actions (see C.3.3.5.1(1)) are similar to those on closed section stiffeners; however the effects of C.3.3.5.1(1) c) are smaller.

|  |  |
| --- | --- |
|  |  |
| **a) with cope holes** | **b) without cope holes** |

Key

|  |  |
| --- | --- |
| 1 | cut-out at bottom of flat to prevent melting of sharp edges |

Figure C.10 — Connections of flat stiffeners with longitudinal welds passing through the web of the crossbeam

* + - * 1. Stiffeners passing through the webs of crossbeams

(1) Closed section stiffeners passing through the web of crossbeams should be designed as follows:

1. without cut-out with welding all around the edge of the stiffeners;
2. or with cut-out around the soffit of the stiffener with partial welding of the stiffener to the web, see Figure C.11 (Haibach cut-out) or Figure C.12 and Figure C.13.

|  |  |
| --- | --- |
|  | *C*U = *h*/3  *C*L = 50 mm  *C*L,min ≥ max{2 *t*w,cb ; 25 mm}  *W*t = 10 mm  *r*1 = variable  *r*u = 20 mm |

Figure C.11 — Webs of crossbeams with Haibach cut-out

(2) Cope holes in the web of the crossbeam at the stiffener deck plate connections should be avoided, see Figure C.12.

(3) The shape of the cut-outs in the web of the crossbeam, see Figure C.12, should be such that:

1. the welds between the stiffeners and the web have adequate strength and should be executed all around the web plates without notches even when crossing the cut-out;
2. the dimensions of the cut-outs allow for stiffener profile tolerances, and surface preparation, application and inspection of the corrosion protection, see Figure C.12 b);
3. this point is a fatigue detail, so the stress range should be checked. Differentiation should be made between road and railway bridges.



Key

|  |  |
| --- | --- |
| 1 | no cope holes, dimension according to Table C.4, lines 3, 4 and 5 |
| 2 | fillet welds |
| 3 | detail for (3) a) |
| 4 | weld around the edge without notches, ground where necessary |
| 5 | detail for (3) b) |
| 6 | detail for (3) c) |

Figure C.12 — Minimum requirements for the design of cut-outs

(4) The minimum size of the cut-out should conform to EN ISO 12944‑3 and Figure C.13.

|  |  |
| --- | --- |
|  | *C*U ≥ 0,15 *h*  *r*u = *C*L  *r*1 = variable  *C*L ≥ max{2 *t*w,cb ; 25 mm} |

Figure C.13 — Minimum dimensions of cut-outs

(5) The requirements for tolerance and inspection are given in Table C.4, line 9.

(6) For the connection of the stiffeners to the end-crossbeam, see C.3.3.5.3.

(7) The requirements for stiffener-crossbeam-connections with stiffeners passing through the crossbeam without cope holes are given in Table C.4, line 8.

* + - * 1. 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;
2. the spacing between crossbeams is ≤ 2,75 m;
3. the steel used for the webs of the crossbeam conforms to the requirements for Z-quality given in prEN 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 Table C.4, line 10.

(3) These requirements also apply to the connections of the stiffeners to the end-crossbeams, irrespective of the spacing between the crossbeams.

* + - * 1. Stiffeners made of flats

(1) Flats should generally pass the webs of the crossbeams and should be welded to the web of the crossbeam on both sides, see Figure C.10.

(2) The slit widths in the crossbeam web should be such that damage due to weld shrinkage is prevented.

(3) The requirements for detailing and inspection are given in Table C.4, line 11.

* + 1. Crossbeams
       1. General

(1) The design of crossbeams should include the following:

1. the required thicknesses of the web plates and the design of the connection of the stiffeners to the crossbeam;
2. the design of the welded connection of the web of the crossbeam to the deck plate
3. the design of the welded connection of the web of the crossbeam to the web of the main girder;
4. the design of the welded connection of the web of the crossbeam to the bottom flange of the crossbeam;
5. the design of the welded 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 the same level;
6. the design of the welded connection of crossbeams to transverse stiffeners, frames or diaphragms positioned in the same plane as the crossbeam.

(2) The design may follow the model given in C.4.5.3.

(3) Any corners of free edges of cut-outs or cope holes should be rounded.

* + - 1. Connections of the web of crossbeam

(1) The requirements and inspection of the welded connections of webs of crossbeams to the deck plate or to the web of the main girder are given in Table C.4, line 12 and Table C.4, line 13, respectively.

(2) Splices of the webs in crossbeams should be welded in accordance with Table C.4, line 14.

* + - 1. Connections of the flange of crossbeams

(1) For continuous crossbeams, 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, line 15.

(2) Where the bottom flanges of the crossbeams and those of the main girders or longitudinal girders are in the same plane, the connections should conform to Table C.4, line 16.

(3) Flange to flange welded joints of crossbeams should conform to Table C.4, line 14.

* + - 1. Transverse stiffeners, frames or diaphragms

(1) In order to reduce local stress concentrations at connections between the crossbeams, transverse stiffeners and diaphragms, suitable 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.14. If this is not the case, fatigue verification should be performed.



Key

|  |  |
| --- | --- |
| 1 | crossbeam |
| 2 | stiffener |
| 3 | transverse stiffener of web of main girder |
| 4 | web of main girder |

Figure C.14 — Typical connection of crossbeams to transverse stiffeners of web of main girders

* 1. Railway bridges
     1. General

(1) This section specifies the structural detailing, weld preparation, execution of the decks of railway bridges to achieve the minimum quality standard required by this document.

(2) Bridge decks of railway bridges may be stiffened as follows:

1. by longitudinal stiffeners and crossbeams;
2. by transverse stiffeners only.

(3) For bridge decks with longitudinal stiffeners, only open section stiffeners made of flats or closed section stiffeners with trapezoidal profiles should be used.

(4) Crossbeams for bridge decks with closed section stiffeners should always be designed with bottom flanges. For bridge decks with longitudinal stiffeners made of flats, crossbeams may be designed without bottom flanges. For bridge decks with transverse stiffeners only, the stiffeners may be used without bottom flanges.

* + 1. Plate thickness and dimensions

(1) For bridge decks with longitudinal stiffeners and crossbeams, see Figure C.15, the dimensions in Table C.1 apply.



Figure C.15 — Notation for the dimensions of bridge decks with longitudinal stiffeners and crossbeams

Table C.1 — Dimensions of bridge decks with longitudinal stiffeners and crossbeams

| Dimensions | Open section stiffeners | Closed section stiffeners |
| --- | --- | --- |
| thickness of deck plate *t*D | *t*D ≥ 14 mm | *t*D ≥ 14 mm |
| spacing *e*LS between stiffeners | 400 mm ≤ *e*LS ≤ 700 mm | 600 mm ≤ *e*LS ≤ 900 mm |
| edge distance *e*E of first stiffener | *e*E ≥ *e*LS | *e*E ≥ *e*LS |
| spacing of crossbeams *e*crossb | *e*crossb ≤ 2700 mm | 2 500 mm ≤ *e*crossb ≤ 3 500 mm |
| ratio of depth of stiffener to depth of crossbeam *h*stiff/*h*crossb | *h*stiff/*h*crossb ≤ 0,5 | *h*stiff/*h*crossb ≤ 0,4 |
| plate thickness *t*stiff | *t*stiff ≥ 10 mm | 6 mm ≤ *t*stiff ≤ 10 mm |
| plate thickness of web of crossbeam *t*w,cb | *t*w,cb ≥ 10 mm | 10 mm ≤ *t*w,cb ≤ 20 mm |
| plate thickness of flange of crossbeam (where flanges are provided) *t*f,cb | *t*f,cb ≥ 10 mm | *t*f,cb ≥ 10 mm |

(2) For bridge decks with only transverse stiffeners, the conditions on dimensions in Table C.2 should be fulfilled.

Table C.2 — Dimensions of bridge deck with only transverse stiffeners

|  |  |
| --- | --- |
| thickness of deck plate *t*D | *t*D ≥ 14 mm |
| spacing of crossbeams *e*crossb | 500 mm ≤ *e*crossb ≤ 800 mm |
| edge distance of crossbeams *e*E | *e*E ≥ 400 mm |
| plate thickness of web crossbeam *t*w,cb | *t*w,cb ≥ 10 mm |
| plate thickness of flange of crossbeam (where flanges are provided) *t*f,cb | *t*f,cb ≥ 10 mm |

* + 1. Design of stiffener to crossbeam connection

(1) Longitudinal stiffeners should 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.16.



Figure C.16 — 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.17.



Key

|  |  |
| --- | --- |
| 1 | weld return, without notches, grinding where necessary |

Figure C.17 — Connection between the closed stiffener and the web of crossbeam

* + 1. Weld preparation and inspections
       1. General

(1) Unless specified otherwise below, Tables C.3 and C.4 apply to detailing, weld preparation, tolerances and inspection.

* + - 1. Weld preparation for stiffener to deck plate connections
         1. Weld preparation of closed section stiffeners

(1) For stiffener plate thicknesses, *t* ≥ 8 mm, (see Table C.4, lines 3 and 4), the edges of the plates should be chamfered as shown in Figure C.18.

(2) For stiffener plate thicknesses, *t* < 8 mm, the edges need not be chamfered if it can be verified by means of welding tests that the requirements for butt welds given in C.4.4.2.2 are met.

* + - * 1. Requirements for butt welds

(1) The requirements for the butt welds are as follows:

seam thickness *a* ≥ 0,9 *t*, where t is the thickness of stiffener;

unwelded gap at root ≤ 0,25 *t* or ≤ 2 mm whichever is the smallest.



Figure C.18 — Weld preparation of stiffener — deck plate connection

* + 1. Analyses
       1. Analysis of longitudinal stiffeners

(1) Longitudinal stiffeners should be analysed as continuous beams on elastic supports.

* + - 1. Analysis of crossbeams - General

(1) For the design of crossbeams, the influence of the cut-outs in the webs of the crossbeams should be taken into account by means of appropriate calculation methods.

* + - 1. Analysis of crossbeams of orthotropic bridge decks with closed section stiffeners

(1) If the crossbeams are executed as illustrated in Figure 10.3, the internal forces and moments may be determined on an equivalent system as shown in Figure 10.3 where the deck plate and parts of the crossbeam below the cut-outs are the flanges and the areas between the cut-outs are the posts.

(2) For calculations in accordance with (1):

* the deformation components due to bending moments, axial forces and shear forces should be taken into account;
* the shear force transmitted from the deck plate to the web of the crossbeam in critical section A-A as shown in Figure 10.4 should be analysed as bending and shear stresses;
* the bending stresses in the areas between the cut-outs referred to above should be superimposed on the compression stresses resulting from the forces transferred locally by the closed stiffeners. When calculating the compression stresses, a load dispersal of 45° should be assumed on the side of the critical section subjected to bending-tensile stresses while a load dispersal of 35° should be assumed on the side of the section subjected to bending-compression stresses;
* the shear stresses in the areas between cut-outs resulting from the horizontal and vertical shear forces should be added together.

(3) The direct stresses in the critical section in Figure 10.4 may be determined using the following Formulae:

where

|  |  |
| --- | --- |
|  | are the normal stresses due to bending; |
|  | are the compressive stresses due to local load from stiffeners. |

|  |  |
| --- | --- |
| *V*Ed = *N*i+1 − *N*i | is the design value of the horizontal shear force |
|  | is the design value of the bending moment in the critical section |
| *F*i, Fi+1 | are the loads introduced from the stiffeners |
| *t*w,cb | is the thickness of the plate |

(4) Transverse bending in the longitudinal welds between closed section stiffeners and deck plate do not need to be verified by calculation, considering that the rules of this annex are followed.

(5) Fatigue verification should be performed according to Clause 10.

* + 1. Flame cut surfaces

(1) Flame-cut surfaces at the cut-outs in crossbeams should conform to quality ranges 1 to 3 in accordance with EN ISO 9013. The edges should be chamfered.

* 1. Tolerances for semi-finished products and fabrication
     1. Tolerances for semi-finished products

(1) Irrespective of the fabrication methods employed for the delivery of semi-finished products, such as 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 the tolerances for semi-finished products that may be used for procurements. This guidance need not be followed where the requirements of Table C.4 are met by other measures.

* + 1. Tolerances for fabrication

(1) The tolerances in Table C.4 apply for the design, fabrication and execution.

(2) The following terms are used in Table C.4:

* Requirement 1: External test results according to quality level of EN ISO 5817;
* Requirement 2: Internal test results according to quality level of EN ISO 5817;
* Requirement 3: See C.5.3;
* Requirement 4: Steels conforming to EN 10164 as specified in prEN 1993‑2.
  + 1. Particular requirements for welded connections

(1) Where required in Table C.4 with reference to this section, the conditions specified in Table C.5 apply in addition to quality level B of EN ISO 5817.

NOTE The following abbreviations for test methods and verifications are used in Table C.4:

1 ultrasonic testing UT

2 radiographic testing RT

3 dye penetration test PT

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 |  |  | Key  1 measure length 2 000 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) | a) | a) and b)    Key  1 max. gap *L*/1 000  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* ≥ 6 mm  For cold forming, only material suitable for cold forming is to be used. *R/t* ≥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. |
| b) | b) |
| 3)  Flat profile for welding on both sides | EN 10029, class C |  |  | Key  1 max. gap L/1000 | Plate thickness *t* ≥ 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    Key  1 misalignment ≤ 2 mm | tensile stress  *σ*Ed ≤ 0,90 *f*y  and  *σ*Ed > 0,75 *f*y | 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 Table C.5. |
| tensile stress  *σ*Ed ≤ 0,75 *f*y  and  *σ*Ed > 0,60 *f*y | 1a Inspection of weld preparation before welding  1b 100 % visual inspection after welding  2 10 % ultrasonic (UT) or radiographic (RT) testing | ad 1a Tolerances for weld preparation to be met, misalignment ≤ 2 mm  ad 1b Requirement 1 and 3  ad 2 Requirement 2 and 3 | Testing requirement, see Table C.5. |
| tensile stress  *σ*Ed ≤ 0,60 *f*y  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 | Testing requirement, see Table C.5. |
| 2)  Splices of deck plate with backing strip    Key  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  *σ*Ed ≤ 0,90 *f*y  and  *σ*Ed > 0,75 *f*y | 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 | re 1a Tolerances for weld preparation to be met,  tack welds of backing strips:  Requirement 1  misalignment ≤ 2 mm  re 1b Requirement 1  fit up gaps between plate and backing strip ≤ 1 mm  re 2 Requirement 2 and 3 | ad 1a Tack weld in the final butt weld,  tack welds with cracks to be removed |
| tensile stress  *σ*Ed ≤ 0,75 *f*y  and  *σ*Ed > 0,60 *f*y | 1a Inspection of weld preparation before welding  1b ≥ 50 % visual inspection after welding  2 10 % radiographic (RT) testing | re 1a Tolerances for weld preparation to be met,  tack welds of backing strips:  Requirement 1  misalignment ≤ 2 mm  re 1b Requirement 1 and 3  re 2 Requirement 2 and 3 | ad 1a Tack weld in the final butt weld,  tack welds with cracks to be removed |
| tensile stress  *σ*Ed ≤ 0,60*f*y  or  compression stress | 1a Inspection of weld preparation before welding  1b 100 % visual inspection after welding | re 1a Tolerances for weld preparation to be met,  misalignment ≤ 2 mm  re 1b Requirement 1 and 3 |  |
| 3)  Stiffener-deck plate 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 a minimum of 1 production test per bridge, with all welding heads, checking by macro section tests. | re 1 Tolerances for weld preparations to be met  re 1b Requirement 1  re 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)  re 3 see ad 2: however macro section tests only from middle of weld of the welding test. | Starts and stops to be removed  re 2 Welding procedure tests under supervision of a recognized body, checking of welding parameters during fabrication  re 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 6). |
| 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 | re 1a Tolerance for weld preparation to be met  re 1b Requirement 1  re 2 Requirement of throat thickness and requirement 1 to be met | Starts and stops to be removed |
| 6)  Stiffener-stiffener connection with splice plates    Key  A site weld  B shop weld | independent on stress level | 1a Inspection of weld preparation before welding  1b ≥ 25 % visual inspection after welding | re 1a Tolerance of gap to be met, misalignment between stiffener and splice plate ≤ 2 mm  re 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.  re 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    Key  1 continuous tack weld  2 misalignment ≤ 2 mm | independent on stress level | 1a Inspection of weld preparation before welding  1b = 25 % visual inspection after welding  2 Test of weld by 1 production test | re 1a Tolerance of weld preparation to be met, misalignment ≤ 2 mm  re 1b Requirement 1  re 2 Requirement 1 and 2 |  |
| b) for plate thicknesses *t* ≥ 8 mm    Key  1 continuous tack weld  2 misalignment ≤ 2 mm  weld preparation angle *α* dependant on welding process and gap width dependant on plate thickness |  |  |  |  |
| 8)  Stiffener-crossbeam connection with stiffeners passing through the crossbeam without cope holes    Key  1 gap ≤ 3 mm | throat thickness  *a* = *a*nom  according to analysis for gap width  *s* ≤ 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 | re 1a Tolerance of weld preparation to be met, required throat thickness *a* available  re 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 cut-outs    Key  1 gap ≤ 3 mm  welds around edges of cut-outs 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 | re 1a Tolerance of weld preparation to be met, required throat thickness *a* available  re 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)a).  3. The cut edges of the webs of crossbeams including the cut-outs should be without notches, in case there are, they should be ground. For flame cutting EN ISO 9013 – Quality 1 applies. |
| 10)  Stiffener-crossbeam connection with stiffeners fitted between crossbeams (not passing through)    Key  1 gap ≤ 2 mm  2 misalignment ≤ 2 mm  single sided full penetration weld (single V-weld) without backing strip | throat thickness  *a* > *t*stiff | 1a Inspection of weld preparation before welding  1b ≥ 50 % visual inspection after welding | re 1a Tolerance of weld preparation to be met, misalignment ≤ 2 mm  re 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. |
| Key  1 stiffener  2 web of crossbeam  3 tack weld  single sided full penetration weld with backing strip |  |  |  |  |
| 11)  Stiffener-crossbeam connection with flats passing through    Key  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 | re 1a Tolerance of weld preparation to be met  re 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)    Key  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 | re 1a Tolerance of weld preparation to be met, requirement 1 and 2  re 1b Requirement 1 | The flame cut edges of the webs of the crossbeams should be prepared in accordance with EN ISO 9013, quality ranges 1‑3. |
| 13)  Connection of webs of crossbeams to web of main girder  a) for continuous crossbeams    Key  1 web of main girder  2 web of crossbeam  3 *t*w,cb  4 misalignment ≤ 0,5 *t*w,cb | independent on stress level | 1a Inspection of weld preparation before welding  1b 100 % visual inspection after welding | re 1a Tolerance of weld preparation to be met, requirement 1 for a), misalignment ≤ 0,5 *t*w,cb. Additional requirements in prEN 1993‑1‑9:2023, Table 10.6 Detail 1, should be considered if relevant.  re 1b Requirement 1 | Execution with full penetration welds, weld preparation angle *α* and weld preparation in accordance with welding process and plate thickness.  A fatigue verification is required for partial penetration welds and double fillet welds. For execution with fillet welds see detail 12. |
| b) for non continuous crossbeams    Key  1 web of main girder  2 web of crossbeam  3 gap ≤ 2 mm | throat thickness of fillet weld according to analysis | see above | re 1a see above  re 1b see above | Execution with fillet welds, see detail 12) |
| 14)  Splice of lower flange or web of crossbeam    Key  1 misalignment 0 mm to 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 | re 1a Tolerance of weld preparation to be met, requirement 1, misalignment ≤ 2 mm  re 1b Requirement 1 and 3  2 Requirement 2 |  |
| 15)  Connection of crossbeam flanges to web of main girder    Key  1 web of main girder  2 web of crossbeam  3 *t*w,cb  4 misalignment ≤ 0,5 *t*w,cb  5 , where *t*w is the web thickness of the main girder. | independent on stress level | 1a Inspection of weld preparation before welding  1b 100 % visual inspection after welding | re 1a Tolerance of weld preparation to be met, misalignment 0,5 ≤ *t*w,cb  re 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. |
| 16)  In plane connection of flanges of crossbeams and main girders    Key  1 main girder  2 crossbeam  3 *b*crossb  4 *b*main girder | minimum radius at connection  min *r* = 150 mm,  Otherwise fatigue verification is required: also applies to continuous crossbeams.  All plate thicknesses are equal. |  |  | Transitions to be ground.  Design with fatigue verification. |

Table C.5 — Conditions supplementary to EN ISO 5817, quality level B

| To No. | Discontinuity | Supplementary requirement |
| --- | --- | --- |
| 3 | Porosity and gas pores | only singular small pores acceptable |
| 4 | Localized (clustered) porosity | maximum volume of pores: 2 % |
| 5 | Gas canal, long pores | no long pores |
| 10 | Bad fit up, fillet welds | test of the complete length of all transverse welds; small root reset only locally acceptable |
|  |  | *r*g ≤ 0,3 + 0,10 *a*,  however *r*g ≤ 1 mm  *r*g = root gap or root reset respectively |
| 11 | Undercut | a) butt welds  only locally acceptable  *h* ≤ 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 |
| 26 | Multiple discontinuities in a cross-section | not acceptable |
| 6 | Solid inclusions | not acceptable |
| 25 | Welding spatter | spatter and their heat affected zones to be removed |

1. (normative)  
     
   Equivalent geometrical imperfections for arched bridges
   1. Use of this annex

(1) This Normative Annex contains additional provisions to 7.3 for the application of equivalent geometrical imperfections for second order analysis according to EN 1993‑1‑1.

* 1. Scope and field of application

(1) This Normative Annex applies to arched bridges.

* 1. Definition of the equivalent geometrical imperfections

(1) Equivalent geometrical imperfections may either be determined from the relevant buckling mode or from simplified equivalent bow imperfections, see EN 1993‑1‑1.

(2) The equivalent bow imperfections given in Table D.1 for in-plane buckling of arches and in Table D.2 for out-of-plane buckling of arches may be used, depending on the buckling curve. The buckling curve should be determined according to Table 8.3 of EN 1993‑1‑1:2022.

NOTE For plate imperfection, see EN 1993‑1‑14.

Table D.1 — Shape and amplitudes of equivalent bow imperfections  
for in-plane buckling of arches

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
|  | Shape of imperfection  (sinus or parabola) | *e*0 according to the buckling curve | | | |
| a | b | c | d |
|  |  |  |  |  |  |
|  |  |  |  |  |  |

Table D.2 — Shape and amplitudes of equivalent bow imperfections   
for out-of-plane buckling of arches

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  | Shape of imperfection  (sinus or parabola) | *e*0 according to the buckling curve | | | | |
|  | a | b | c | d |
|  |  | *L* ≤ 20 m |  |  |  |  |
| For *L* > 20 m,  Where *L* and *L*1 are in m |  |  |  |  |

1. (normative)  
     
   Combination of effects from local wheel and tyre loads and from global traffic loads on road bridges
   1. Use of this annex

(1) This Normative Annex contains additional provisions to 4.2.1 for the combination of effects from local wheel and tyre loads and from global traffic loads.

* 1. Scope and field of application

(1) This Normative Annex applies to road bridges.

* 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) In case different load models are used for calculating the global and the local effects, to take into account the different sources of these loads, the combination rule according to the Formulae (E.1) and (E.2) may be applied to determine the design values:

|  |  |
| --- | --- |
|  | (E.1) |
|  | (E.2) |

where

|  |  |
| --- | --- |
| *σ*Ed | is the design value of stress in the stiffener due to combined effects of local load *σ*loc,Ed and global load *σ*glob,Ed; |
| *σ*loc.Ed | is the design value of stress in the stiffener due to local wheel or tyre load from a single heavy vehicle; |
| *σ*glob.Ed | is the design value of stress in the stiffener 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 *σ*loc.Ed



c) Analysis model to determine global effects *σ*glob.Ed

Figure E.1 — Modelling of structure with local and global effects

* 1. 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 combination factors to be used are those in Figure E.2, unless the National Annex gives different values. The National Annex can also give guidance on the combination factor depending on the load models used.



Figure E.2 — Combination factor dependent on span length *L*

1. (informative)  
     
   Damage equivalent factors *λ* for fatigue verification of road bridge decks
   1. Use of this annex

(1) This Informative Annex provides the damage equivalent factors for the fatigue verification of road bridge decks using the damage equivalent factors method.

(2) Two different sets of damage equivalent factors *λ* may be adopted by using Clause F.2 or Clause F.3.

NOTE The National choice on the application of Clause F.2 or Clause F.3 of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, both clauses can be used.

* 1. Set 1 of damage equivalent factors *λ*

### F2.1 Scope and field of application

(1) This set of damage equivalent factors *λ* applies to road bridges with critical span lengths between 5 m and 80 m.

(2) For critical span lengths outside the range given in (1), Clause F.3 or the fatigue load models 4 or 5 should be used.

### F2.2 Simplified fatigue load model

(1) For the use of this set of damage equivalent factors *λ*, the fatigue load model 3 defined in EN 1991‑2 should be applied.

### F2.3 Damage equivalent factors *λ*

(1) The factor *λ*1 should be determined according to Figure F.1, classifying the bridge deck section from Figure 10.7 and calculating the critical length *L*λ of the influence line or area using the rules in (2).

(2) The critical length *L*λ of the influence line or area should be determined as follows:

1. for bending moments:

* for a simply supported span, the span length *L*i;
* for continuous spans in midspan sections, the span length *L*i of the span under consideration;
* for continuous spans in support sections, the mean of the two spans *L*i and *L*j adjacent to that support;
* for cross girders supporting stiffeners, the sum of the two adjacent spans of the stiffeners carried by the cross girder.

1. for shear forces:

* for the support section, the span under consideration *L*i;
* for the midspan section, 0,4 × the span under consideration *L*i.

1. for support reactions:

* for the end support, the span under consideration *L*i;
* for the intermediate support, the sum of the two spans *L*i and *L*j adjacent to that support.

1. for arch bridges:

* for hangers, twice the distance between hangers;
* for arch, half the span of the arch.

Dimensions in m

|  |  |
| --- | --- |
|  |  |
| a) Midspan section | b) Support section |

Figure F.1 — Factors *λ*1 for bending moments for road bridges

(3) The factor *λ*2 should be calculated using Formula (F.1):

|  |  |
| --- | --- |
|  | (F.1) |

where

|  |  |  |
| --- | --- | --- |
|  | is the average gross weight (kN) of the lorries in the slow lane obtained from Formula (F.2): | |
|  | | (F.2) |
| *Q*0 = 480 kN | | |
| *N*0 = 0,5 × 106 | | |
| *N*Obs | is the total number of lorries per year in the slow lane, according to EN 1991‑2, and specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties; | |
| *Qi* | is the gross weight in kN of the lorry *i* in the slow lane as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties; | |
| *ni* | is the number of lorries of gross weight *Q*i in the slow lane as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties. | |

(4) For given values of and , factor may be obtained from Table F.1.

Table F.1 — Factor *λ*2 for road bridges

| *Q*m1 | *N*Obs | | | | | | | |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| 0,25 × 106 | 0,50 × 106 | 0,75 × 106 | 1,00 × 106 | 1,25 × 106 | 1,50 × 106 | 1,75 × 106 | 2,00 × 106 |
| 200 | 0,36 | 0,42 | 0,45 | 0,48 | 0,50 | 0,52 | 0,54 | 0,55 |
| 300 | 0,54 | 0,63 | 0,68 | 0,71 | 0,75 | 0,78 | 0,80 | 0,83 |
| 400 | 0,73 | 0,83 | 0,90 | 0,96 | 1,00 | 1,04 | 1,07 | 1,10 |
| 500 | 0,91 | 1,04 | 1,13 | 1,20 | 1,25 | 1,30 | 1,34 | 1,37 |
| 600 | 1,09 | 1,25 | 1,36 | 1,44 | 1,50 | 1,56 | 1,61 | 1,65 |

(5) The factor *λ*3 should be calculated using Formula (F.3) or obtained from Table F.2, for a given design service life of the bridge in years:

|  |  |
| --- | --- |
|  | (F.3) |

Table F.2 — Factor *λ*3 for road bridges

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| Design service life in years | 50 | 60 | 70 | 80 | 90 | 100 | 120 |
| Factor *λ*3 | 0,87 | 0,90 | 0,93 | 0,96 | 0,98 | 1,00 | 1,04 |

NOTE The design service life of the bridge is *t*Ld = 100 years, see EN 1990:2023, Clause A.2.

(6) The factor *λ*4 should be calculated using Formula (F.4):

|  |  |
| --- | --- |
|  | (F.4) |

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 the Formula (F.4) with positive sign. |

(7) The factor should be obtained from the relevant stress-range spectrum. It may be determined from Figure F.2.

Dimensions in m

|  |  |
| --- | --- |
|  |  |
| a) Midspan section | b) Support section |

Figure F.2 — Factors *λ*max for bending moments for road bridges

* 1. Set 2 of damage equivalent factors *λ*

### F3.1 Scope and field of application

(1) This set of damage equivalent factors applies to road bridges with critical span lengths between 5 m and 200 m.

(2) For critical span lengths outside the range given in (1), load models 4 or 5 should be used.

### F3.2 Simplified fatigue load model for road bridges

(1) For the use of this set of damage equivalent factors , only the first vehicle of fatigue load model 3 defined in EN 1991‑2 (single vehicle model with four axles each weighting 120 kN) should be applied.

### F3.3 Damage equivalent factors *λ*

(1) The factor λ1 should be determined according to Figure F.3 and Table F.3, classifying the bridge deck section from Figure 10.7 and calculating the critical length of the influence line or area using the rules in (2).

NOTE The factors *λ*1 were obtained using a series of fatigue resistance curves with slopes = 3 and = 5, according to EN 1993‑1‑9, and can be used for the other fatigue resistance curves.

Dimensions in m

|  |  |
| --- | --- |
|  |  |
| For m:  For m: | For m:  For m: |
| a) Midspan section bending moment (MM) | b) Support section bending moment (SM) |
|  |  |
| For  m:  For  m: | For  m:  For  m: |
| c) Midspan section shear force (MV) | d) Support section shear force and reaction (SV) |

Figure F.3 — Factors *λ*1 for road bridge deck sections

Table F.3 — Factors *λ*1 for road bridge deck sections

|  | ***λ*1** | | | |
| --- | --- | --- | --- | --- |
| ***L*λ** | **MM** | **SM** | **MV** | **SV** |
| 5 | 1,74 | 1,71 | 1,50 | 1,60 |
| 6 | 1,72 | 1,67 | 1,47 | 1,58 |
| 7 | 1,69 | 1,64 | 1,45 | 1,57 |
| 8 | 1,67 | 1,61 | 1,43 | 1,55 |
| 9 | 1,65 | 1,58 | 1,40 | 1,53 |
| 10 | 1,63 | 1,56 | 1,38 | 1,52 |
| 12 | 1,60 | 1,52 | 1,34 | 1,49 |
| 15 | 1,56 | 1,50 | 1,30 | 1,47 |
| 17 | 1,55 | 1,49 | 1,27 | 1,46 |
| 20 | 1,54 | 1,51 | 1,24 | 1,46 |
| 25 | 1,55 | 1,54 | 1,28 | 1,48 |
| 30 | 1,56 | 1,58 | 1,31 | 1,50 |
| 35 | 1,56 | 1,61 | 1,35 | 1,52 |
| 40 | 1,57 | 1,64 | 1,38 | 1,53 |
| 45 | 1,58 | 1,67 | 1,40 | 1,55 |
| 50 | 1,59 | 1,69 | 1,43 | 1,57 |
| 60 | 1,60 | 1,75 | 1,47 | 1,61 |
| 70 | 1,62 | 1,80 | 1,51 | 1,64 |
| 80 | 1,63 | 1,85 | 1,54 | 1,68 |
| 90 | 1,65 | 1,90 | 1,57 | 1,72 |
| 100 | 1,66 | 1,95 | 1,59 | 1,76 |
| 120 | 1,69 | 2,04 | 1,63 | 1,83 |
| 140 | 1,72 | 2,12 | 1,66 | 1,90 |
| 160 | 1,75 | 2,20 | 1,70 | 1,97 |
| 180 | 1,78 | 2,28 | 1,74 | 2,03 |
| 200 | 1,81 | 2,36 | 1,81 | 2,08 |

(2) The critical length of the influence line or area should be determined as follows:

1. for bending moments:

* for a simply supported span, the span length ;
* for continuous spans in midspan sections, the span length of the span under consideration;
* for continuous spans in support sections, the mean of the two spans and adjacent to that support;
* for double spans bridge decks in the intermediate support section, the sum of the two spans and , if ;
* for cross girders supporting stiffeners, the sum of the two adjacent spans of the stiffeners carried by the cross girder.

1. for shear forces:

* the span under consideration .

1. for support reactions:

* for the end support, the span under consideration ;
* for the intermediate support, the sum of the two spans and adjacent to that support.

1. for arch bridges:

* for hangers, twice the distance between hangers;
* for arch, half the span of the arch.

(3) The factor *λ*2 should be calculated using Formula (F.5):

|  |  |
| --- | --- |
|  | (F.5) |

where

|  |  |  |
| --- | --- | --- |
| *Q*m1 | is the average gross weight (kN) of the lorries in the slow lane obtained from Formula (F.6): | |
|  | | (F.6) |
| *Q*0 = 350 kN | | |
| *N*0 = 0,5 × 106 | | |
| = 5 | | |
| *N*Obs | is the total number of lorries per year in the slow lane, according to EN 1991‑2, and specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties; | |
| *Qi* | is the gross weight in kN of the lorry *i* in the slow lane as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties; | |
| *ni* | is the number of lorries of gross weight *Q*i in the slow lane as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties. | |

(4) For given values of *Q*m1 and *N*Obs , factor *λ*2 may be obtained from Table F.4.

Table F.4 — Factor *λ*2 for road bridges

| *Q*m1 | *N*Obs | | | | | | | | | |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| 0,05 × 106 | 0,1 × 106 | 0,25 × 106 | 0,50 × 106 | 0,75 × 106 | 1,00 × 106 | 1,25 × 106 | 1,50 × 106 | 1,75 × 106 | 2,00 × 106 |
| 200 | 0,36 | 0,41 | 0,50 | 0,57 | 0,62 | 0,66 | 0,69 | 0,71 | 0,73 | 075 |
| 300 | 0,54 | 0,62 | 0,75 | 0,86 | 0,93 | 0,99 | 1,03 | 1,07 | 1,10 | 1,13 |
| 400 | 0,72 | 0,83 | 1,00 | 1,14 | 1,24 | 1,31 | 1,37 | 1,42 | 1,47 | 1,51 |
| 500 | 0,90 | 1,04 | 1,24 | 1,43 | 1,55 | 1,64 | 1,72 | 1,78 | 1,84 | 1,89 |
| 600 | 1,08 | 1,24 | 1,49 | 1,71 | 1,86 | 1,97 | 2,06 | 2,14 | 2,20 | 2,26 |

(5) The factor *λ*3 should be calculated using Formula (F.7) or obtained from Table F.5, for a given design service life of the bridge in years and = 5:

|  |  |
| --- | --- |
|  | (F.7) |

Table F.5 — Factor *λ*3 for road bridges

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| Design service life in years | 50 | 60 | 70 | 80 | 90 | 100 | 120 |
| Factor *λ*3 | 0,87 | 0,90 | 0,93 | 0,96 | 0,98 | 1,00 | 1,04 |

NOTE The design service life of the bridge is *t*Ld = 100 years, see EN 1990:2023, Clause A.2.

(6) The factor *λ*4 should be calculated using Formula (F.8):

|  |  |
| --- | --- |
|  | (F.8) |

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 the Formula (F.8) with positive sign; |
| = 5. | |

(7) The factor should be determined according to Figure F.4 and Table F.6, classifying the bridge deck section from Figure 10.7, taking into account the adopted in (3) and calculating the critical length of the influence line or area using the rules in (2).

NOTE The factors *λ*max were obtained using a series of fatigue resistance curves with slopes  = 3 and  = 5, and a constant fatigue limit *N*D = 5 million cycles according to EN 1993‑1‑9 can be used for the other fatigue resistance curves.

Dimensions in m

|  |  |
| --- | --- |
|  |  |
| Equations of the curve for :  For m:  For m: | Equations of the curve for :  For m:  For m: |
| a) Midspan section bending moment (MM) | b) Support section bending moment (SM) |
|  |  |
| Equations of the curve for :  For m:  For m: | Equations of the curve for :  For m:  For m: |
| c) Midspan section shear force (MV) | d) Support section shear force and reaction (SV) |

Figure F.4 — Factors *λ*max for road bridge deck sections

Table F.6 — Factors *λ*max for road bridge deck sections

|  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | a) *λ*max for *N*Obs = 0,05 × 106 | | | |  |  | b) *λ*max for *N*Obs = 0,125 × 106 | | | |
| *L*λ | MM | SM | MV | SV |  | *L*λ | MM | SM | MV | SV |
| 5 | 1,40 | 1,40 | 1,11 | 1,22 |  | 5 | 1,68 | 1,68 | 1,34 | 1,47 |
| 6 | 1,38 | 1,38 | 1,09 | 1,21 |  | 6 | 1,66 | 1,65 | 1,31 | 1,45 |
| 7 | 1,36 | 1,35 | 1,07 | 1,19 |  | 7 | 1,63 | 1,63 | 1,28 | 1,43 |
| 8 | 1,34 | 1,34 | 1,05 | 1,18 |  | 8 | 1,61 | 1,61 | 1,26 | 1,41 |
| 9 | 1,32 | 1,32 | 1,03 | 1,17 |  | 9 | 1,59 | 1,59 | 1,24 | 1,40 |
| 10 | 1,31 | 1,31 | 1,02 | 1,15 |  | 10 | 1,57 | 1,57 | 1,22 | 1,39 |
| 12 | 1,28 | 1,28 | 0,99 | 1,14 |  | 12 | 1,54 | 1,54 | 1,19 | 1,37 |
| 15 | 1,25 | 1,27 | 0,97 | 1,12 |  | 15 | 1,50 | 1,52 | 1,16 | 1,35 |
| 17 | 1,24 | 1,26 | 0,96 | 1,12 |  | 17 | 1,49 | 1,52 | 1,15 | 1,35 |
| 20 | 1,24 | 1,27 | 0,97 | 1,13 |  | 20 | 1,49 | 1,53 | 1,16 | 1,36 |
| 25 | 1,24 | 1,32 | 1,00 | 1,15 |  | 25 | 1,49 | 1,58 | 1,20 | 1,38 |
| 30 | 1,25 | 1,36 | 1,02 | 1,17 |  | 30 | 1,50 | 1,64 | 1,23 | 1,41 |
| 35 | 1,25 | 1,40 | 1,05 | 1,19 |  | 35 | 1,51 | 1,69 | 1,26 | 1,44 |
| 40 | 1,26 | 1,44 | 1,08 | 1,22 |  | 40 | 1,51 | 1,73 | 1,30 | 1,46 |
| 45 | 1,27 | 1,48 | 1,10 | 1,24 |  | 45 | 1,52 | 1,78 | 1,33 | 1,49 |
| 50 | 1,28 | 1,52 | 1,13 | 1,27 |  | 50 | 1,54 | 1,83 | 1,35 | 1,52 |
| 60 | 1,30 | 1,59 | 1,17 | 1,32 |  | 60 | 1,56 | 1,91 | 1,41 | 1,59 |
| 70 | 1,32 | 1,66 | 1,21 | 1,38 |  | 70 | 1,59 | 1,99 | 1,45 | 1,65 |
| 80 | 1,34 | 1,72 | 1,25 | 1,43 |  | 80 | 1,61 | 2,07 | 1,50 | 1,72 |
| 90 | 1,37 | 1,78 | 1,28 | 1,49 |  | 90 | 1,64 | 2,14 | 1,53 | 1,79 |
| 100 | 1,40 | 1,84 | 1,31 | 1,55 |  | 100 | 1,68 | 2,21 | 1,57 | 1,86 |
| 120 | 1,45 | 1,95 | 1,36 | 1,67 |  | 120 | 1,75 | 2,34 | 1,63 | 2,01 |
| 140 | 1,51 | 2,04 | 1,40 | 1,79 |  | 140 | 1,82 | 2,45 | 1,68 | 2,15 |
| 160 | 1,57 | 2,13 | 1,43 | 1,90 |  | 160 | 1,89 | 2,56 | 1,72 | 2,28 |
| 180 | 1,63 | 2,22 | 1,47 | 2,00 |  | 180 | 1,96 | 2,66 | 1,77 | 2,40 |
| 200 | 1,69 | 2,30 | 1,51 | 2,08 |  | 200 | 2,03 | 2,76 | 1,81 | 2,50 |
|  | c) *λ*max for *N*Obs = *N*0 = 0,5 × 106 | | | |  |  | d) *λ*max for *N*Obs = 2,0 × 106 | | | |
| *L*λ | MM | SM | MV | SV |  | *L*λ | MM | SM | MV | SV |
| 5 | 2,22 | 2,2 | 1,76 | 1,94 |  | 5 | 2,93 | 2,92 | 2,33 | 2,56 |
| 6 | 2,19 | 2,18 | 1,73 | 1,91 |  | 6 | 2,89 | 2,88 | 2,28 | 2,52 |
| 7 | 2,15 | 2,15 | 1,69 | 1,89 |  | 7 | 2,84 | 2,83 | 2,23 | 2,49 |
| 8 | 2,12 | 2,12 | 1,66 | 1,87 |  | 8 | 2,80 | 2,80 | 2,19 | 2,46 |
| 9 | 2,10 | 2,09 | 1,63 | 1,85 |  | 9 | 2,76 | 2,76 | 2,16 | 2,44 |
| 10 | 2,07 | 2,07 | 1,61 | 1,83 |  | 10 | 2,73 | 2,73 | 2,12 | 2,41 |
| 12 | 2,03 | 2,03 | 1,57 | 1,80 |  | 12 | 2,67 | 2,68 | 2,07 | 2,38 |
| 15 | 1,98 | 2,01 | 1,53 | 1,78 |  | 15 | 2,62 | 2,65 | 2,02 | 2,35 |
| 17 | 1,97 | 2,00 | 1,52 | 1,78 |  | 17 | 2,59 | 2,64 | 2,01 | 2,34 |
| 20 | 1,96 | 2,02 | 1,53 | 1,79 |  | 20 | 2,59 | 2,67 | 2,02 | 2,36 |
| 25 | 1,97 | 2,09 | 1,58 | 1,82 |  | 25 | 2,60 | 2,76 | 2,08 | 2,41 |
| 30 | 1,98 | 2,16 | 1,62 | 1,86 |  | 30 | 2,61 | 2,85 | 2,14 | 2,45 |
| 35 | 1,99 | 2,22 | 1,67 | 1,89 |  | 35 | 2,62 | 2,93 | 2,20 | 2,50 |
| 40 | 2,00 | 2,29 | 1,71 | 1,93 |  | 40 | 2,64 | 3,02 | 2,26 | 2,55 |
| 45 | 2,01 | 2,35 | 1,75 | 1,97 |  | 45 | 2,65 | 3,10 | 2,31 | 2,60 |
| 50 | 2,03 | 2,41 | 1,79 | 2,01 |  | 50 | 2,67 | 3,18 | 2,36 | 2,65 |
| 60 | 2,06 | 2,52 | 1,86 | 2,09 |  | 60 | 2,71 | 3,33 | 2,44 | 2,76 |
| 70 | 2,09 | 2,63 | 1,92 | 2,18 |  | 70 | 2,76 | 3,47 | 2,53 | 2,88 |
| 80 | 2,13 | 2,73 | 1,97 | 2,27 |  | 80 | 2,81 | 3,60 | 2,61 | 3,00 |
| 90 | 2,17 | 2,83 | 2,02 | 2,37 |  | 90 | 2,86 | 3,73 | 2,67 | 3,12 |
| 100 | 2,21 | 2,92 | 2,07 | 2,46 |  | 100 | 2,92 | 3,85 | 2,73 | 3,25 |
| 120 | 2,30 | 3,08 | 2,15 | 2,65 |  | 120 | 3,04 | 4,07 | 2,83 | 3,50 |
| 140 | 2,40 | 3,24 | 2,21 | 2,84 |  | 140 | 3,17 | 4,26 | 2,92 | 3,74 |
| 160 | 2,49 | 3,38 | 2,27 | 3,01 |  | 160 | 3,29 | 4,45 | 3,00 | 3,97 |
| 180 | 2,59 | 3,51 | 2,33 | 3,17 |  | 180 | 3,41 | 4,64 | 3,07 | 4,18 |
| 200 | 2,67 | 3,65 | 2,39 | 3,30 |  | 200 | 3,53 | 4,81 | 3,15 | 4,35 |

(3) The value of may be obtained directly for any using Formula (F.9), with given in Table F.6 for *N*0 = 0,5 × 106 and  = 5:

|  |  |
| --- | --- |
|  | (F.9) |

Bibliography

**References contained in recommendations (i.e. through “should” clauses)**

The following documents are referred to in the text in such a way that some or all of their content, although not requirements strictly to be followed, constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative standards could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1337 (all parts), *Structural bearings*

EN 10025 (all parts), *Hot rolled products of structural steels*

EN 10149 (all parts), *Hot rolled flat products made of high yield strength steels for cold forming*

EN 10164, *Steel products with improved deformation properties perpendicular to the surface of the product — Technical delivery conditions*

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EN 15129, *Anti-seismic devices*

EN ISO 9013, *Thermal cutting — Classification of thermal cuts — Geometrical product specification and quality tolerances*

**References contained in permissions (i.e. through “may” clauses)**

The following documents are referred to in the text in such a way that some or all of their content, although not requirements strictly to be followed, expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

CEN/TS 1993‑1‑101, *Eurocode 3 — Design of steel structures — Part 1‑101: Alternative interaction method for members in bending and compression*

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*Tension members susceptible to vibration in bridge construction — Design rules for hangers on tied arch bridges and recommendations for fatigue-proof construction (Schwingungsanfällige Zugglieder im Brückenbau — Bemessungsregeln für Hänger an Stabbogenbrücken und Empfehlungen für ermüdungsgerechtes Konstruieren), version 02/2018,* download via [www.baw.de](file:///C:\Users\wilkinst\AppData\Local\Microsoft\Windows\INetCache\Content.Outlook\9V5737EN\www.baw.de) or <http://www.bast.de>.

1. As impacted by EN 1990:2023/prA1:2024. [↑](#footnote-ref-1)
2. Under preparation. [↑](#footnote-ref-2)
3. Under preparation. [↑](#footnote-ref-3)