

The European Union

EDICT OF GOVERNMENT

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EN 1993-6 (2007) (English): Eurocode 3: Design of steel structures - Part 6: Crane supporting structures
[Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]



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English Version

Eurocode 3 - Design of steel structures - Part 6: Crane supporting structures

Eurocode 3 - Calcul des structures en acier - Partie 6:
Chemins de roulement

Eurocode 3 - Bemessung und Konstruktion von
Stahlbauten - Teil 6: Kranbahnen

This European Standard was approved by CEN on 12 June 2006.

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Foreword

This European Standard EN 1993-6, “Eurocode 3: Design of steel structures: Part 6 Crane supporting structures”, has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI.
CEN/TC250 is responsible for all Structural Eurocodes.

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of an identical text or by endorsement, at the latest by October 2007, and conflicting National Standards shall be withdrawn at latest by March 2010.

This Eurocode supersedes ENV 1993-6.

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

Background of the Eurocode programme

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

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

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

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to the CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products – CPD – and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

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

- EN 1990 Eurocode: Basis of structural design
- EN 1991 Eurocode 1: Actions on structures
- EN 1992 Eurocode 2: Design of concrete structures
- EN 1993 Eurocode 3: Design of steel structures
- EN 1994 Eurocode 4: Design of composite steel and concrete structures
- EN 1995 Eurocode 5: Design of timber structures
- EN 1996 Eurocode 6: Design of masonry structures
- EN 1997 Eurocode 7: Geotechnical design
- EN 1998 Eurocode 8: Design of structures for earthquake resistance
- EN 1999 Eurocode 9: Design of aluminium structures

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

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

Status and field of application of Eurocodes

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

as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 - Mechanical resistance and stability - and Essential Requirement N°2 - Safety in case of fire;

as a basis for specifying contracts for construction works and related engineering services;

as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standard³. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving a full compatibility of these technical specifications with the Eurocodes.

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

National Standards implementing Eurocodes

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

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

values and/or classes where alternatives are given in the Eurocode,

values to be used where a symbol only is given in the Eurocode,

country specific data (geographical, climatic etc.) e.g. snow map,

the procedure to be used where alternative procedures are given in the Eurocode,

– references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

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

- give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
- indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;
- serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

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

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works⁴. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1993-6

EN 1993-6 is one of the six parts of EN 1993 "Design of Steel Structures" and gives principles and application rules for the safety, serviceability and durability of crane supporting structures.

EN 1993-6 gives design rules that supplement the generic rules in EN 1993-1.

EN 1993-6 is intended for clients, designers, contractors and public authorities.

EN 1993-6 is intended to be used with EN 1990, EN 1991 and EN 1993-1. Matters that are already covered in those documents are not repeated.

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

National Annex for EN 1993-6

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may be made. So the National Standard implementing EN 1993-6 should have a National Annex containing all Nationally Determined Parameters to be used for the design of crane-supporting members in steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-6 through:

2.1.3.2(1)P	Design working life.
2.8(2)P	Partial factor $\gamma_{F, \text{test}}$ for crane test loads.
3.2.3(1)	Lowest service temperature for indoor crane supporting structures.
3.2.3(2)P	Selection of toughness properties for members in compression.
3.2.4(1) table 3.2	Requirement Z_{Ed} for through-thickness properties.
3.6.2(1)	Information on suitable rails and rail steels.
3.6.3(1)	Information on special connecting devices for rails.
6.1(1)	Partial factors γ_{Mi} for resistance for ultimate limit states.
6.3.2.3(1)	Alternative assessment method for lateral-torsional buckling
7.3(1)	Limits for deflections and deformations.
7.5(1)	Partial factor $\gamma_{M, \text{ser}}$ for resistance for serviceability limit states.
8.2(4)	Crane classes to be treated as "high fatigue".
9.1(2)	Limit for number of cycles C_0 without a fatigue assessment.
9.2(1)P	Partial factors γ_{Ff} for fatigue loads.
9.2(2)P	Partial factors γ_{Mf} for fatigue resistance.
9.3.3(1)	Crane classes where bending due to eccentricity may be neglected.
9.4.2(5)	Damage equivalence factors λ_{dup} for multiple crane operation.

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

1 General

1.1 Scope

- (1) This Part 6 of EN 1993 provides design rules for the structural design of runway beams and other crane supporting structures.
- (2) The provisions given in Part 6 supplement, modify or supersede the equivalent provisions given in EN 1993-1.
- (3) It covers overhead crane runways inside buildings and outdoor crane runways, including runways for:
- a) overhead travelling cranes, either:
 - supported on top of the runway beams;
 - underslung below the runway beams;
 - b) monorail hoist blocks.
- (4) Additional rules are given for ancillary items including crane rails, structural end stops, support brackets, surge connectors and surge girders. However, crane rails not mounted on steel structures, and rails for other purposes, are not covered.
- (5) Cranes and all other moving parts are excluded. Provisions for cranes are given in EN 13001.
- (6) For seismic design, see EN 1998.
- (7) For resistance to fire, see EN 1993-1-2.

1.2 Normative references

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

EN 1090 *Execution of steel structures and aluminium structures:*

Part 2 *Technical requirements for steel structures;*

EN 1337 *Structural bearings;*

EN ISO 1461 *Hot dip galvanised coatings on fabricated iron and steel articles – specifications and test methods;*

EN 1990 *Eurocode: Basis of structural design;*

EN 1991 *Eurocode 1: Actions on structures:*

Part 1-1 *Actions on structures – Densities, self-weight and imposed loads for buildings;*

Part 1-2 *Actions on structures – Actions on structures exposed to fire;*

Part 1-4 *Actions on structures – Wind loads;*

Part 1-5 *Actions on structures – Thermal actions;*

Part 1-6 *Actions on structures – Construction loads;*

Part 1-7 *Actions on structures – Accidental actions;*

Part 3 *Actions on structures – Actions induced by cranes and machinery;*

EN 1993 *Eurocode 3: Design of steel structures:*

Part 1-1 *General rules and rules for buildings;*

Part 1-2 *Structural fire design;*

Part 1-4 *Stainless steels;*

Part 1-5 *Plated structural elements;*

Part 1-8: *Design of joints;*

Part 1-9: *Fatigue;*

Part 1-10: *Material toughness and through thickness properties;*

EN 1998 *Eurocode 8: Design provisions for earthquake resistance of structures;*

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

ISO/DIS 11660 *Cranes - Access, guards and restraints:*

Part 5 *Bridge and gantry cranes.*

TS 13001 *Cranes - General design;*

Part 3.3 *Limit states and proof of competence of wheel/rail contacts;*

1.3 Assumptions

(1) In addition to the general assumptions of EN 1990 the following assumptions apply:

- **AC1** execution **AC1** complies with EN 1090-2.

1.4 Distinction between principles and application rules

(1) See 1.4 in EN 1990.

1.5 Terms and definitions

(1) See 1.5 in EN 1993-1-1.

(2) Supplementary to EN 1991-3, for the purposes of this Part 6 the following terminology applies:

1.5.1 crane surge Horizontal dynamic actions due to crane operation, acting longitudinally and/or laterally to the runway beams.

NOTE: The transverse actions induced by cranes apply lateral forces to the runway beams.

1.5.2 elastomeric bearing pad Resilient reinforced elastomeric bedding material intended for use under crane rails.

1.5.3 surge connector Connection that transmits crane surge from a runway beam, or a surge girder, to a support.

1.5.4 surge girder Beam or lattice girder that resists crane surge and carries it to the supports.

1.5.5 structural end stop. Component intended to stop a crane or hoist reaching the end of a runway.

1.6 Symbols

(1) The symbols are defined in EN 1993-1-1 and where they first occur in this EN 1993-6.

NOTE: The symbols used are based on ISO 3898: 1987.

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1) See 2.1.1 of EN 1993-1-1.

2.1.2 Reliability management

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

2.1.3 Design working life, durability and robustness

2.1.3.1 General

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

2.1.3.2 Design working life

(1)P The design working life of a crane supporting structure shall be specified as the period during which it is required to provide its full function. The design working life should be documented (for example in the maintenance plan).

NOTE: The National Annex may specify the relevant design working life. A design working life of 25 years is recommended for runway beams, but for runways that are not intensively used, 50 years may be appropriate.

(2)P For temporary crane supporting structures, the design working life shall be agreed with the client and the public authority, taking account of possible re-use.

(3) For structural components that cannot be designed to achieve the total design working life of the crane supporting structure, see 4(6).

2.1.3.3 Durability

(1)P Crane supporting structures shall be designed for environmental influences, such as corrosion, wear and fatigue by appropriate choice of materials, see EN 1993-1-4 and EN 1993-1-10, appropriate detailing, see EN 1993-1-9, structural redundancy and appropriate corrosion protection.

(2)P Where replacement or realignment is necessary (e.g. due to expected soil subsidence) such replacement or realignment shall be taken into account in the design by appropriate detailing and verified as a transient design situation.

2.2 Principles of limit state design

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

2.3 Basic variables

2.3.1 Actions and environmental influences

(1)P The characteristic values of crane actions shall be determined by reference to EN 1991-3.

NOTE 1: EN 1991-3 gives rules for determining crane actions in accordance with the provisions in EN 13001-1 and EN 13001-2 to facilitate the exchange of data with crane suppliers.

NOTE 2: EN 1991-3 gives various methods to determine reliable actions, depending upon whether or not full information on crane specifications are available at the time of design of crane supporting structures.

(2)P Other actions on crane supporting structures shall be determined by reference to EN 1991-1-1, EN 1991-1-2, EN 1991-1-4, EN 1991-1-5, EN 1991-1-6 or EN 1991-1-7 as appropriate.

(3)P Partial factors and combination rules shall be taken from Annex A of EN 1991-3.

(4) For actions during erection stages see EN 1991-1-6.

(5) For actions from soil subsidence see 2.3.1(3) and (4) of EN 1993-1-1.

2.3.2 Material and product properties

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

2.4 Verification by the partial factor method

(1) See 2.4 of EN 1993-1-1.

(2) For partial factors for static equilibrium and uplift of bearings see Annex A of EN 1991-3.

2.5 Design assisted by testing

(1) See 2.5 of EN 1993-1-1.

2.6 Clearances to overhead travelling cranes

(1) The clearances between all overhead travelling cranes and the crane supporting structure, and the dimensions of all access routes to the cranes for drivers or for maintenance personnel, should comply with ISO/DIS 11660-5.

2.7 Underslung cranes and hoist blocks

(1) $\boxed{AC_1}$ Where the bottom flange $\boxed{AC_1}$ of a runway beam directly supports wheel loads from an underslung crane or hoist block, a serviceability limit state stress check, see 7.5, should be carried out.

(2) The ultimate limit state resistance of this flange should also be verified as specified in 6.7.

2.8 Crane tests

(1) Where a crane or a hoist block is required to be tested after erection on its supporting structure, a serviceability limit state stress check, see 7.5, should be carried out on the supporting members affected, using the relevant crane test loads from 2.10 of EN 1991-3.

(2)P The ultimate limit state verifications specified in 6 shall also be satisfied under the crane test loads, applied at the positions affected. A partial factor $\gamma_{F, \text{test}}$ shall be applied to these test loads.

NOTE: The numerical value for $\boxed{AC_1}$ $\gamma_{F, \text{test}}$ $\boxed{AC_1}$ may be defined in the National Annex. The value of 1,1 is recommended.

3 Materials

3.1 General

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

3.2 Structural steels

3.2.1 Material properties

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

3.2.2 Ductility requirements

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

3.2.3 Fracture toughness

(1) See 3.2.3(1) and (2) of EN 1993-1-1.

NOTE: The lowest service temperature to be adopted in design for indoor crane supporting structures may be given in the National Annex.

(2)P For components under compression a suitable minimum toughness property shall be selected.

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

(3) For the choice of steels suitable for cold forming (e.g. for pre-cambering) and subsequent hot dip zinc coating see EN 1461.

3.2.4 Through thickness properties

(1) See 3.2.4(1) of EN 1993-1-1.

NOTE 1: Particular care should be given to welded beam-to-column connections and welded end plates with tension in the through-thickness direction.

NOTE 2: The National Annex may specify the allocation of target values Z_{Ed} according to 3.2(3) of EN 1993-1-10 to the quality class in EN 10164. The allocation in table 3.2 is recommended for crane supporting structures.

Table 3.2 Choice of quality class according to EN 10164

Target value of Z_{Ed} according to EN 1993-1-10	Required value of Z_{Rd} according to EN 10164
≤ 10	—
11 to 20	Z 15
21 to 30	Z 25
> 30	Z 35

3.2.5 Tolerances

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

3.2.6 Design values of material coefficients

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

3.3 Stainless steels

(1) For stainless steels see the relevant provisions in EN 1993-1-4.

3.4 Fasteners and welds

(1) See 3.3 of EN 1993-1-1.

3.5 Bearings

(1) Bearings should comply with EN 1337.

3.6 Other products for crane supporting structures

3.6.1 General

(1) Any semi-finished or finished structural product used in the structural design of a crane supporting structure should comply with the relevant EN Product Standard or ETAG or ETA.

3.6.2 Rail steels

(1) Purpose-made crane rails and railway rails should both be made from special rail steels, with a specified minimum tensile strengths of between 500 N/mm² and 1200 N/mm².

NOTE: The National Annex may give information for suitable rails and rail steels, pending the issue of appropriate product specifications (EN product standards, ETAGs or ETAs).

(2) $\overline{\text{AC1}}$ Rectangular bars $\overline{\text{AC1}}$ and other sections used as rails may also be of structural steels as specified in 3.2.

3.6.3 Special connecting devices for rails

(1) Special connecting devices for rails, including purpose made fixings and elastomeric bearing pads should be suitable for their specific use according to the relevant product specifications.

NOTE: The National Annex may give information for special connecting devices, where no appropriate product specification (EN product standard, ETAG or ETA) exists.

4 Durability

(1) For durability of steel structures generally, see 4(1), 4(2) and 4(3) of EN 1993-1-1.

(2) For crane supporting structures fatigue assessments should be carried out according to section 9.

(3) Where crane rails are assumed to contribute to the strength or stiffness of a runway beam, appropriate allowances for wear should be made in determining the properties of the combined cross-section, see 5.6.2(2) and 5.6.2(3).

(4) Where actions from soil subsidence or seismic actions are expected, tolerances for vertical and horizontal imposed deformations should be agreed with the crane supplier and included in the inspection and maintenance plans.

(5) The expected values of imposed deformations should be taken into account by appropriate detailing for readjustment.

(6) Structural components that cannot be designed with sufficient reliability to achieve the total design working life of the crane supporting structure, should be replaceable. Such parts may be:

- expansion joints,
- crane rails and their fixings,
- elastomeric bearing pads,
- surge connections.

5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

- (1) See 5.1.1(1), (2) and (3) of EN 1993-1-1.
- (2) See also EN 1993-1-5 for shear lag effects and plate buckling.

5.1.2 Joint modelling

- (1) See 5.1.2 (1), (2) and (3) of EN 1993-1-1.
- (2) The modelling of joints that are subject to fatigue should be such that sufficient fatigue life can be verified according to EN 1993-1-9.

NOTE: In crane supporting structures, bolts acting in shear in bolted connections where the bolts are subject to forces that include load reversals, should either be fitted bolts or else be preloaded bolts designed to be slip-resistant at ultimate limit state, Category C of EN 1993-1-8.

5.1.3 Ground structure interaction

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

5.2 Global analysis

5.2.1 Effects of deformed geometry of the structure

- (1) See 5.2.1 of EN 1993-1-1.

5.2.2 Structural stability of frames

- (1) See 5.2.2 of EN 1993-1-1.

5.3 Imperfections

5.3.1 Basis

- (1) See 5.3.1 of EN 1993-1-1.

5.3.2 Imperfections for global analysis of frames

- (1) See 5.3.2 of EN 1993-1-1.
- (2) The imperfections for global analysis need not be combined with the eccentricities given in 2.5.2.1(2) of EN 1991-3.

5.3.3 Imperfections for analysis of bracing systems

- (1) See 5.3.3 of EN 1993-1-1.

5.3.4 Member imperfections

- (1) See 5.3.4 of EN 1993-1-1.
- (2) The member imperfections need not be combined with the eccentricities given in 2.5.2.1(2) of EN 1991-3.

5.4 Methods of analysis

5.4.1 General

- (1) See 5.4.1 of EN 1993-1-1.
- (2) In crane supporting structures where fatigue resistance is required, elastic global analysis is recommended. If plastic global analysis is used for the ultimate limit state verification of a runway beam, a serviceability limit state stress check should also be carried out, see 7.5.

5.4.2 Elastic global analysis

- (1) See 5.4.2 of EN 1993-1-1.

5.4.3 Plastic global analysis

- (1) See 5.4.3 and 5.6 of EN 1993-1-1.

5.5 Classification of cross-sections

(1) See 5.5 of EN 1993-1-1.

5.6 Runway beams

5.6.1 Effects of crane loads

(1) The following internal forces and moments due to crane loads should be taken into account in the design of runway beams:

biaxial bending due to vertical actions and lateral horizontal actions;

axial compression or tension due to longitudinal horizontal actions;

torsion due to the eccentricity of lateral horizontal actions, relative to the shear centre of the cross-section of the beam;

- vertical and horizontal shear forces due to vertical actions and lateral horizontal actions.

(2) In addition, local effects due to wheel loads should be taken into account.

5.6.2 Structural system

(1) If a crane rail is rigidly fixed to the top flange of the runway beam, by means of fitted bolts, preloaded bolts in Category C connections (designed to be non-slip at ultimate limit states, see 3.4.1 of EN 1993-1-8) or by welding, it may be included as part of the cross-section that is taken into account to calculate the resistance. Such bolts or welds should be designed to resist the longitudinal shear forces arising from bending due to vertical and horizontal actions, together with the forces due to horizontal crane actions.

(2) To allow for wear, the nominal height of the rail should be reduced when calculating the cross-section properties. This reduction should generally be taken as 25 % of the minimum nominal thickness t_r below the wearing surface, see figure 5.1, unless otherwise stated in the maintenance plan, see 4(3).

(3) For fatigue assessments only half of the reduction given in (2) need be made.

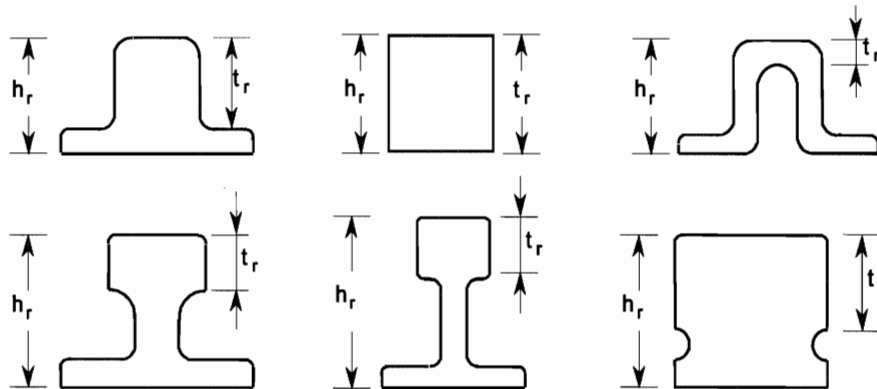


Figure 5.1: Minimum thickness t_r below the wearing surface of a crane rail

(4) Except when box sections are used, it may be assumed that crane loads are resisted as follows:

vertical wheel loads are resisted by the main vertical beam located under the rail;

lateral loads from top-mounted cranes are resisted by the top flange or surge girder;

lateral loads from underslung cranes or hoist blocks are resisted by the bottom flange;

(a) torsional moments are resisted by couples acting horizontally on the top and bottom flanges.

(5) Alternatively to (4), the effects of torsion may be treated as in EN 1993-1-1.

(6) In-service wind loads F_W^* and lateral horizontal crane loads $H_{T,3}$ due to acceleration or braking of the crab hoist block should be assumed to be shared between the runway beams in proportion to their lateral stiffnesses if the crane has doubly-flanged wheels, but should all be applied to the runway beams on one side if the crane uses guide rollers.

5.7 Local stresses in the web due to wheel loads on the top flange

5.7.1 Local vertical compressive stresses

(1) The local vertical compressive stress $\sigma_{oz,Ed}$ generated in the web by wheel loads on the top flange, see figure 5.2 may be determined from:

$$\sigma_{oz,Ed} = \frac{F_{z,Ed}}{\ell_{eff} t_w} \quad (5.1)$$

where: $F_{z,Ed}$ is the design value of the wheel load;

ℓ_{eff} is the effective loaded length;

t_w is the thickness of the web plate.

(2) The effective loaded length ℓ_{eff} over which the local vertical stress $\sigma_{oz,Ed}$ due to a single wheel load may be assumed to be uniformly distributed, may be determined using table 5.1. Crane rail wear in accordance with 5.6.2(2) and 5.6.2(3) should be taken into account.

(3) If the distance x_w between the centres of adjacent crane wheels is less than ℓ_{eff} the stresses from the two wheels should be superposed.

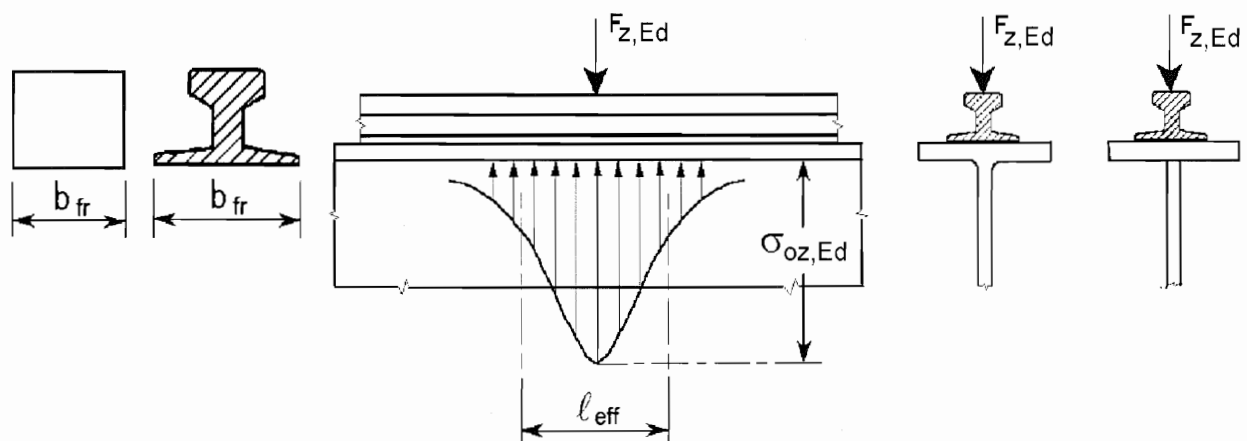


Figure 5.2: Effective loaded length ℓ_{eff}

(4) The local vertical stress $\sigma_{0z,Ed}$ at other levels in the web may be calculated by assuming a further distribution at each wheel load at 45° from the effective loaded length ℓ_{eff} at the underside of the top flange, see figure 5.3, provided that if the total length of dispersion exceeds the distance x_w between adjacent wheels, the stresses from the two wheels are superposed.

(5) Remote from the supports, the local vertical stress $\sigma_{0z,Ed}$ calculated using this length should be multiplied by the reduction factor $[1 - (z/h_w)^2]$ where h_w is the overall depth of the web and z is the distance below the underside of the top flange, see figure 5.3.

(6) Close to the supports, the local vertical compressive stress due to a similar dispersion of the support reaction should also be determined and the larger value of the stress $\sigma_{0z,Ed}$ adopted.

Table 5.1: Effective loaded length ℓ_{eff}

Case	Description	Effective loaded length ℓ_{eff}
(a)	Crane rail rigidly fixed to the flange	$\ell_{eff} = 3,25 [I_{rf} / t_w]^{1/3}$
(b)	Crane rail not rigidly fixed to flange	$\ell_{eff} = 3,25 [(I_r + I_{f,eff}) / t_w]^{1/3}$
(c)	Crane rail mounted on a suitable resilient elastomeric bearing pad at least 6mm thick.	$\ell_{eff} = 4,25 [(I_r + I_{f,eff}) / t_w]^{1/3}$
$I_{f,eff}$ is the second moment of area, about its horizontal centroidal axis, of a flange with an effective width of b_{eff} I_r is the second moment of area, about its horizontal centroidal axis, of the rail I_{rf} is the second moment of area, about its horizontal centroidal axis, of the combined cross-section comprising the rail and a flange with an effective width of b_{eff} t_w is the web thickness.		
$b_{eff} = b_{fr} + h_r + t_f$ but $b_{eff} \leq b$ where: b is the overall width of the top flange; b_{fr} is the width of the foot of the rail, see figure 5.2; h_r is the height of the rail, see figure 5.1; t_f is the flange thickness.		
Note: Allow for crane rail wear, see 5.6.2(2) and 5.6.2(3) in determining I_r , I_{rf} and h_r .		

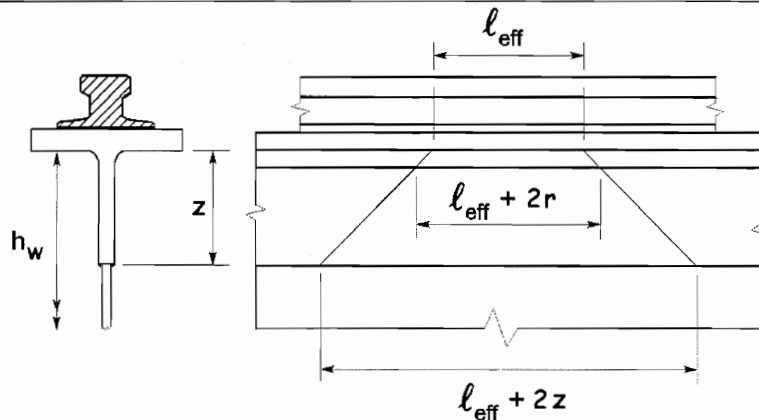


Figure 5.3: Distribution at 45° from effective loaded length ℓ_{eff}

5.7.2 Local shear stresses

(1) The maximum value of the local shear stress $\tau_{0xz,Ed}$ due to a wheel load, acting at each side of the wheel load position, may be assumed to be equal to 20% of the maximum local vertical stress $\sigma_{0z,Ed}$ at that level in the web.

(2) The local shear stress $\tau_{0xz,Ed}$ at any point should be taken as additional to the global shear stress due to the same wheel load, see figure 5.4. The additional shear stress $\tau_{0xz,Ed}$ may be neglected at levels in the web below $z = 0,2h_w$, where h_w and z are as defined in 5.7.1(5).

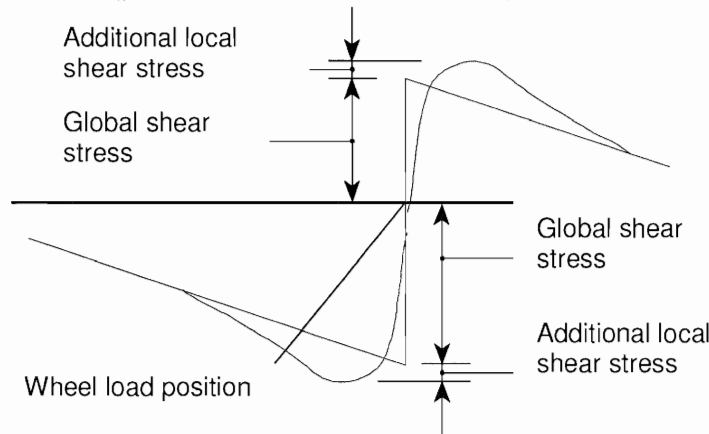


Figure 5.4: Local and global shear stresses due to a wheel load

5.7.3 Local bending stresses in the web due to eccentricity of wheel loads

(1) The bending stress $\sigma_{T,Ed}$ in a transversely stiffened web due to the torsional moment may be determined from:

$$\sigma_{T,Ed} = \frac{6 T_{Ed}}{a t_w^2} \eta \tanh(\eta) \quad (5.2)$$

$$\text{with: } \eta = \left[\frac{0,75 a t_w^3}{I_t} \times \frac{\sinh^2(\pi h_w / a)}{\sinh(2 \pi h_w / a) - 2 \pi h_w / a} \right]^{0,5} \quad (5.3)$$

where: a is the spacing of the transverse web stiffeners;

h_w is the overall depth of the web, clear between flanges;

I_t is the torsion constant of the flange (including the rail if it is rigidly fixed).

(2) The torsional moment T_{Ed} due to the lateral eccentricity e_y of each wheel load $F_{z,Ed}$, see figure 5.5, should be obtained from:

$$T_{Ed} = F_{z,Ed} e_y \quad (5.4)$$

where: e_y is the eccentricity e of the wheel load given in 2.5.2.1(2) of EN 1991-3,
but $e_y \geq 0,5 t_w$,

t_w is the thickness of the web.

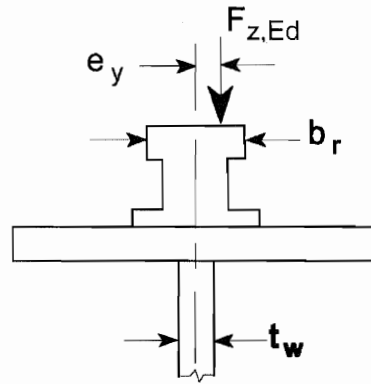


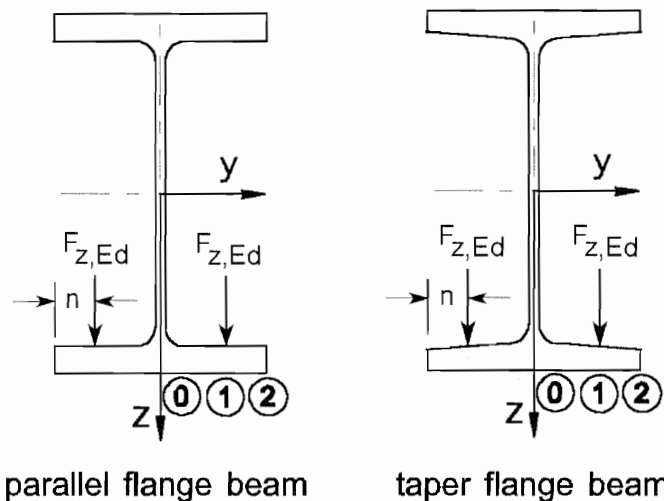
Figure 5.5: Torsion of the top flange

5.8 Local bending stresses in the bottom flange due to wheel loads

(1) The following method may be used to determine the local bending stresses in the bottom flange of an I-section beam, due to wheel loads applied to the bottom flange.

(2) The bending stresses due to wheel loads applied at locations more than b from the end of the beam, where b is the flange width, can be determined at the three locations indicated in figure 5.6:

- - location 0: the web-to-flange transition;
- - location 1: centreline of the wheel load;
- - location 2: outside edge of the flange.



parallel flange beam

taper flange beam

Figure 5.6: Locations for determining stresses due to wheel loads

(3) Provided that the distance x_w along the runway beam between adjacent wheel loads is not less than $1,5b$, where b is the flange width of the beam, the local longitudinal bending stress $\sigma_{ox,Ed}$ and transverse bending stress $\sigma_{oy,Ed}$ in the bottom flange due to the application of a wheel load more than b from end of the beam should be obtained from:

$$\sigma_{ox,Ed} = c_x F_{z,Ed} / t_1^2 \quad (5.5)$$

$$\sigma_{oy,Ed} = c_y F_{z,Ed} / t_1^2 \quad (5.6)$$

where: $F_{z,Ed}$ is the vertical crane wheel load;

t_1 is the thickness of the flange at the centreline of the wheel load.

(4) Generally the coefficients c_x and c_y for determining the longitudinal and transverse bending stresses at the three locations **0**, **1** and **2** shown in figure 5.6 may be determined from table 5.2 depending on whether the beam has parallel flanges or taper flanges, and the value of the ratio μ given by:

$$\mu = 2n / (b - t_w) \quad (5.7)$$

where: n is the distance from the centreline of the wheel load to the free edge of the flange;

t_w is the thickness of the web.

Table 5.2: Coefficients c_{xi} and c_{yi} for calculating stresses at points $i = 0, 1$ and 2

Stress	Parallel flange beams	Taper flange beams (See Note)
Longitudinal bending stress $\sigma_{ox,Ed}$	$c_{x0} = 0,050 - 0,580\mu + 0,148e^{3,015\mu}$	$c_{x0} = -0,981 - 1,479\mu + 1,120e^{1,322\mu}$
	$c_{x1} = 2,230 - 1,490\mu + 1,390e^{-18,33\mu}$	$c_{x1} = 1,810 - 1,150\mu + 1,060e^{-7,700\mu}$
	$c_{x2} = 0,730 - 1,580\mu + 2,910e^{-6,000\mu}$	$c_{x2} = 1,990 - 2,810\mu + 0,840e^{-4,690\mu}$
Transverse bending stress $\sigma_{oy,Ed}$	$c_{y0} = -2,110 + 1,977\mu + 0,0076e^{6,530\mu}$	$c_{y0} = -1,096 + 1,095\mu + 0,192e^{-6,000\mu}$
	$c_{y1} = 10,108 - 7,408\mu - 10,108e^{-1,364\mu}$	$c_{y1} = 3,965 - 4,835\mu - 3,965e^{-2,675\mu}$
	$c_{y2} = 0,0$	$c_{y2} = 0,0$
Sign convention: $c_{x,i}$ and $c_{y,i}$ are positive for tensile stresses at the bottom face of the flange.		
NOTE: The coefficients for taper flange beams are for a slope of 14% or 8°. They are conservative for beams with a larger flange slope. For beams with a smaller flange slope, it is conservative to adopt the coefficients for parallel flange beams. Alternatively linear interpolation may be used.		

(5) Alternatively, in the case of wheel loads applied near the outside edges of the flange, the values of the coefficients c_x and c_y given in table 5.3 may be used.

Table 5.3: Coefficients for calculating stresses near the outside edges of flanges

Stress	Coefficient	Parallel flange beams		Taper flange beams (See Note)
		$\mu = 0,10$	$\mu = 0,15$	$\mu = 0,15$
Longitudinal bending stress $\sigma_{ox,Ed}$	c_{x0}	0,2	0,2	0,2
	c_{x1}	2,3	2,1	2,0
	c_{x2}	2,2	1,7	2,0
Transverse bending stress $\sigma_{oy,Ed}$	c_{y0}	-1,9	-1,8	-0,9
	c_{y1}	0,6	0,6	0,6
	c_{y2}	0,0	0,0	0,0
Sign convention: $c_{x,i}$ and $c_{y,i}$ are positive for tensile stresses at the bottom face of the flange.				
NOTE: The coefficients for taper flange beams are for a slope of 14% or 8°. They are conservative for beams with a larger flange slope. For beams with a smaller flange slope, it is conservative to adopt the coefficients for parallel flange beams. Alternatively linear interpolation may be used.				

(6) In the absence of better information, the local bending stress $\sigma_{oy,end,Ed}$ in an unstiffened bottom flange due to the application of wheel loads at a perpendicular end of the beam should be determined from:

$$\sigma_{oy,end,Ed} = (5,6 - 3,225\mu - 2,8\mu^3) F_{z,Ed} / t_f \quad (5.8)$$

where: t_f is the mean thickness of the flange.

(7) Alternatively, if the bottom flange is reinforced at the end by welding on a plate of similar thickness extending across its width b and for a distance of at least b along the beam, see figure 5.7, the local bending stress $\sigma_{oy,end,Ed}$ may be assumed not to exceed $\sigma_{ox,Ed}$ and $\sigma_{oy,Ed}$ from (3) or (5).

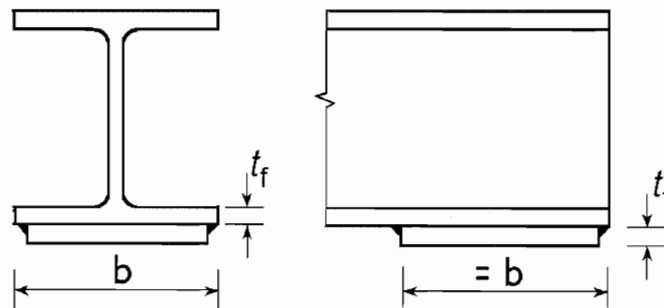


Figure 5.7: Optional reinforcement at the end of the bottom flange

(8) If the distance x_w between adjacent wheel loads is less than $1,5b$, a conservative approach may be adopted by superposing the stresses calculated for each wheel load acting separately, unless special measures (such as testing, see 2.5) are adopted to determine the local stresses.

5.9 Secondary moments in triangulated components

(1) Secondary moments due to joint rigidity in members of lattice girders, lattice surge girders and triangulated bracing panels may be allowed for using $k1$ -factors as specified in 4(2) of EN 1993-1-9.

(2) For members of open cross-section the $k1$ -factors given in table 5.4 may be used.

(3) For members made from structural hollow sections with welded joints, the $k1$ -factors given in table 4.1 and table 4.2 of EN 1993-1-9 may be used.

Table 5.4: Coefficients k_1 for secondary stresses in members of open cross-section

(a) Lattice girders loaded only at nodes			
Range of L/y values	$L/y \leq 20$	$20 < L/y < 50$	$L/y \geq 50$
Chord members	1,57	$\frac{1,1}{0,5 + 0,01 L / y}$	1,1
End and internal members			
Secondary members, see Note	1,35	1,35	1,35
(b) Lattice girders with chord members loaded between nodes			
Range of L/y values	$L/y < 15$	$L/y \geq 15$	
Loaded chord members	$\frac{0,4}{0,25 + 0,01 L / y}$	1,0	
Unloaded chord members	1,35	1,35	
Secondary members, see Note			
End members	2,50	2,50	
Internal members	1,65	1,65	
Key:			
L is the length of the member between nodes;			
y is the perpendicular distance, in the plane of triangulation, from the centroidal axis of the member to its relevant edge, measured, as follows:			
<ul style="list-style-type: none">- compression chord: in the direction from which the loads are applied;- tension chord: in the direction in which the loads are applied;- other members: the larger distance.			
NOTE: Secondary members comprise members provided to reduce the buckling lengths of other members or to transmit applied loads to nodes. In an analysis assuming hinged joints, the forces in secondary members are not affected by loads applied at other nodes, but in practice they are affected due to joint rigidity and the continuity of chord members at joints.			

6 Ultimate limit states

6.1 General

(1) The partial factors γ_M for resistance apply to the various characteristic values in section 6 as indicated in table 6.1.

Table 6.1 Partial factors for resistance

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

Note: The partial factors γ_{Mi} for crane supporting structures may be defined in the National Annex. The following numerical values are recommended:

γ_{M0}	=	1,00
γ_{M1}	=	1,00
γ_{M2}	=	1,25
γ_{M3}	=	1,25
$\gamma_{M3,ser}$	=	1,10
γ_{M4}	=	1,00
γ_{M5}	=	1,00
$\gamma_{M6,ser}$	=	1,00
γ_{M7}	=	1,10

6.2 Resistance of cross-section

(1) See 6.2 of EN 1993-1-1.

6.3 Buckling resistance of members

6.3.1 General

(1) See 6.3 of EN 1993-1-1.

6.3.2 Lateral-torsional buckling

6.3.2.1 General

(1) In checking the lateral-torsional buckling resistance of a runway beam, the torsional moments due to the eccentricities of vertical actions and lateral horizontal actions relative to the shear centre should be taken into account.

NOTE: The methods given in 6.3 of EN 1993-1-1 do not cover torsional moments.

6.3.2.2 Effective level of application of wheel loads

(1) If the crane wheel loads are applied to a runway beam through a rail without an elastomeric bearing pad, allowance may be made for the stabilizing effect of the horizontal shift in the point of application of the vertical wheel reaction to the rail, that occurs when there is torsional rotation. Provided that the cross-section of the beam is a plain or lipped I-section, in the absence of a more precise analysis it may be assumed to be conservative to take the vertical wheel reaction as being effectively applied at the level of the shear centre.

(2) If the crane wheel loads are applied through a rail supported on an elastomeric bearing pad, or applied directly to the top flange of a runway beam, the simplification detailed in (1) should not be relied upon, and the vertical wheel reaction should be taken as being effectively applied at the level of the top of the flange.

(3) In the case of wheel loads from a monorail hoist block or an underslung crane, the stabilizing effect of applying the loads to the bottom flange should be allowed for. However due to the possible effects of swinging hoist loads, in the absence of a more precise analysis the vertical reaction should not be taken as being effectively applied below the level of the top surface of the bottom flange.

6.3.2.3 Assessment methods

(1) The lateral torsional buckling resistance of a simply supported runway beam may be verified by checking the compression flange plus one fifth of the web against flexural buckling as a compression member. It should be checked for an axial compressive force equal to the bending moment due to the vertical actions, divided by the depth between the centroids of the flanges. The bending moment due to the lateral horizontal actions should also be taken into account, together with the effects of torsion.

NOTE: The National Annex may specify alternative assessment methods. The method given in Annex A is recommended.

6.4 Built up compression members

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

6.5 Resistance of the web to wheel loads

6.5.1 General

(1) The web of a runway beam supporting a top-mounted crane should be checked for resistance to the transverse forces applied by the crane wheel loads.

(2) In this check, the effects of the lateral eccentricity of the wheel loads may be neglected.

(3) The resistance of the web of a rolled or welded section to a transverse force applied $\overline{AC_1}$ through a top flange AC_1 should be determined using section 6 of EN 1993-1-5.

(4) For the interaction of transverse forces with moments and axial force, see 7.2 in EN 1993-1-5.

6.5.2 Length of stiff bearing

(1) The length of stiff bearing s_s on the upper surface of the top flange, due to a crane wheel load applied through a rail, to be used in 6.5 of EN 1993-1-5, may be obtained by using:

$$s_s = \ell_{\text{eff}} - 2 t_f \quad (6.1)$$

where: ℓ_{eff} is the effective loaded length at the underside of the top flange, from table 5.1;

t_f is the thickness of the top flange.

6.6 Buckling of plates

(1) For buckling $\overline{\lambda}_{\text{cr}}$ of plates in sections $\overline{\lambda}_{\text{cr}}$ the rules 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 one of the following methods:

- resistances to design direct stresses, shear stresses and transverse forces are determined according to section 4, 5 or 6 respectively of EN 1993-1-5, and combined using the appropriate interaction formulae in section 7 of EN 1993-1-5,
- the resistance is determined on the basis of class 3 cross-sections with stress limits governed by local buckling according to section 10 of EN 1993-1-5.

(3) For stiffeners in stiffened plates loaded in compression which receive additional bending moments from loads transverse to the plane of the stiffened plate, the stability may be verified according to 6.3.3 of EN 1993-1-1.

6.7 Resistance of bottom flanges to wheel loads

(1) The design resistance $F_{f,Rd}$ of the bottom flange of a beam to a wheel load $F_{z,Ed}$ from an underslung crane or hoist block trolley wheel, see figure 6.1, should be determined from:

$$F_{f,Rd} = \frac{\ell_{\text{eff}} t_f^2 f_y / \gamma_{M0}}{4 m} \left[1 - \left(\frac{\sigma_{f,Ed}}{f_y / \gamma_{M0}} \right)^2 \right] \quad (6.2)$$

where: ℓ_{eff} is the effective length of flange resisting the wheel load, see (3);

m is the lever arm from the wheel load to the root of the flange, see (2);

t_f is the flange thickness;

$\sigma_{f,Ed}$ is the stress at the midline of the flange due to the overall internal moment in the beam.

(2) The lever arm m from the wheel load to the root of the flange should be determined as follows:

$$\text{for a rolled section} \quad m = 0,5 (b - t_w) - 0,8 r - n \quad (6.3)$$

$$\text{for a welded section} \quad m = 0,5 (b - t_w) - 0,8 \sqrt{2} a - n \quad (6.4)$$

where: a is the throat size of a fillet weld;

b is the flange width;

n is the distance from the centreline of the wheel load to the edge of the flange;

r is the root radius;

t_w is the web thickness.

(3) The effective length of flange ℓ_{eff} resisting one wheel load should be determined from table 6.2.

Table 6.2: Effective length ℓ_{eff}

Case	Wheel position	ℓ_{eff}
(a)	Wheel adjacent to a non-reinforced simple joint	$2(m+n)$
(b)	Wheel remote from the end of a member	$4\sqrt{2}(m+n)$ for $x_w \geq 4\sqrt{2}(m+n)$
		$2\sqrt{2}(m+n) + 0,5x_w$ for $x_w < 4\sqrt{2}(m+n)$
(c)	Wheel adjacent to an end stop at a distance $x_e \leq 2\sqrt{2}(m+n)$ from the end of the member	$2(m+n) \left[\frac{x_e}{m} + \sqrt{1 + \left(\frac{x_e}{m} \right)^2} \right]$ but $\leq \sqrt{2}(m+n) + x_e$ for $x_w \geq 2\sqrt{2}(m+n) + x_e$
		$2(m+n) \left[\frac{x_e}{m} + \sqrt{1 + \left(\frac{x_e}{m} \right)^2} \right]$ but $\leq \sqrt{2}(m+n) + \frac{x_w + x_e}{2}$ for $x_w < 2\sqrt{2}(m+n) + x_e$
(d)	Wheel adjacent to an end that is fully supported either from below or by a welded closer plate, see figure 6.2, at a distance $x_e \leq 2\sqrt{2}(m+n)$ from the end of the member	$2\sqrt{2}(m+n) + x_e + \frac{2(m+n)^2}{x_e}$ for $x_w \geq 2\sqrt{2}(m+n) + x_e + \frac{2(m+n)^2}{x_e}$
		$\sqrt{2}(m+n) + \frac{(x_e + x_w)}{2} + \frac{(m+n)^2}{x_e}$ for $x_w < 2\sqrt{2}(m+n) + x_e + \frac{2(m+n)^2}{x_e}$

where: x_e is the distance from the end of member to the centreline of the wheel;

x_w is the wheel spacing.

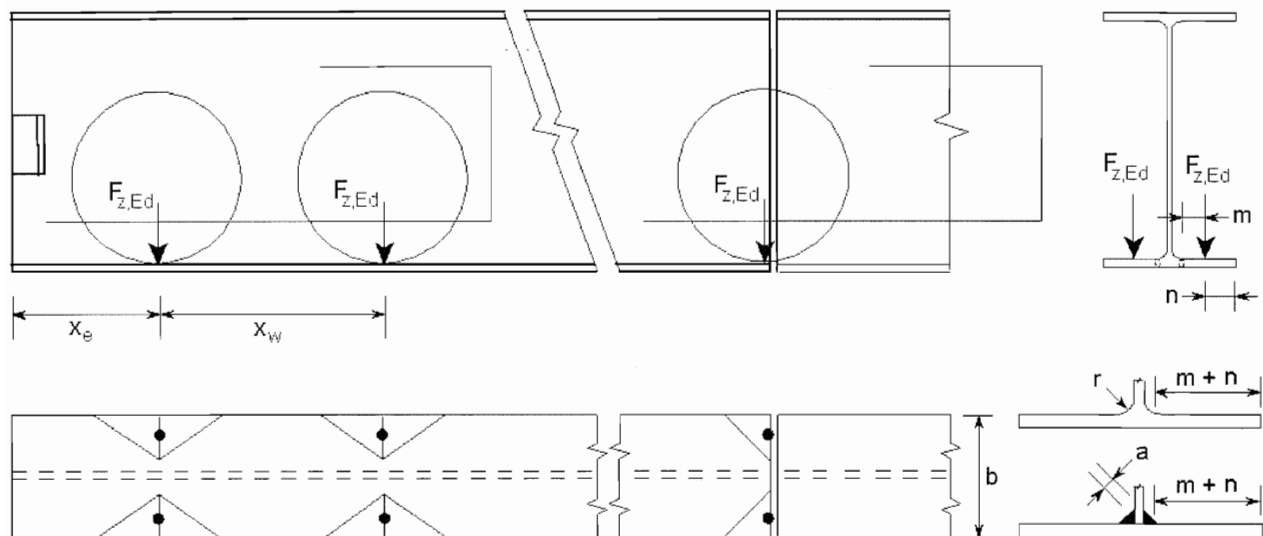


Figure 6.1: Bending of bottom flange remote from ends and at non-reinforced joints

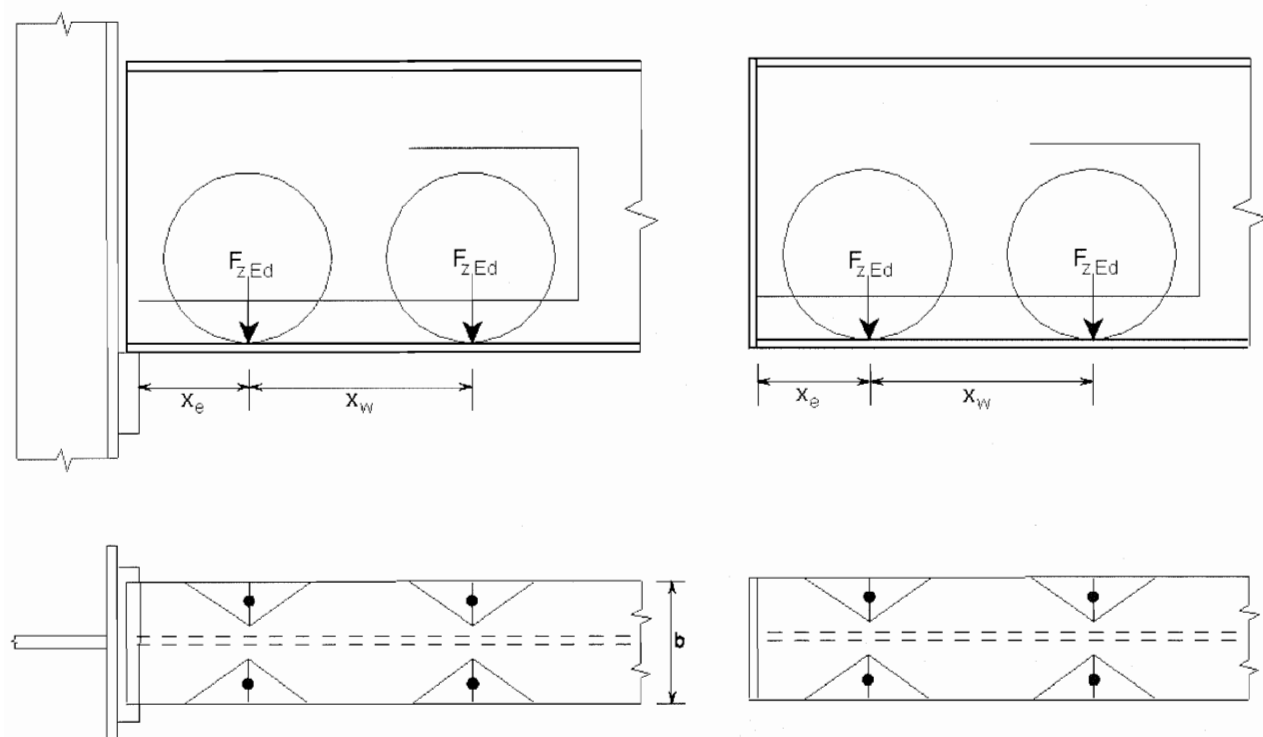


Figure 6.2: Bending of bottom flange at fully supported ends

7 Serviceability limit states

7.1 General

(1) In addition to the ultimate limit state criteria, the following serviceability limit state criteria should also be satisfied:

a) deformations and displacements, see **7.3**:

- vertical deformation of runway beams, to avoid excessive vibrations caused by hoist or crane operation or travel;
- vertical deformation of runway beams, to avoid excessive slope of the runway;
- differential vertical deformation of a pair of runway beams, to avoid excessive slope of the crane;
- horizontal deformation of runway beams, to reduce skewing of the crane;
- lateral displacement of supporting columns or frames at crane support level, to avoid excessive amplitude of frame vibrations;
- differential lateral displacement of adjacent columns or frames, to avoid abrupt changes in horizontal alignment of crane rails, causing increased skewing and possible distortion of crane bridges;
- lateral movements that change the spacing of a pair of crane beams, to avoid damage to wheel flanges, rail fixings or crane structures;

b) plate slenderness, in order to exclude visible buckling or breathing of web plates, see **7.4**;

c) stresses, in order to ensure reversible behaviour, see **7.5**:

- where wheels are supported directly on the flange of a runway beam, see **2.7**;
- under crane test loading (from 2.10 of EN 1991-3), see **2.8(1)**;
- where plastic global analysis is used for the ultimate limit state verification, see **5.4.1(2)**.

7.2 Calculation models

(1) Stresses and displacements at serviceability limit states should be determined by linear elastic analysis, see EN 1993-1-1.

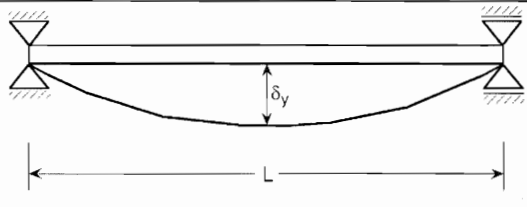
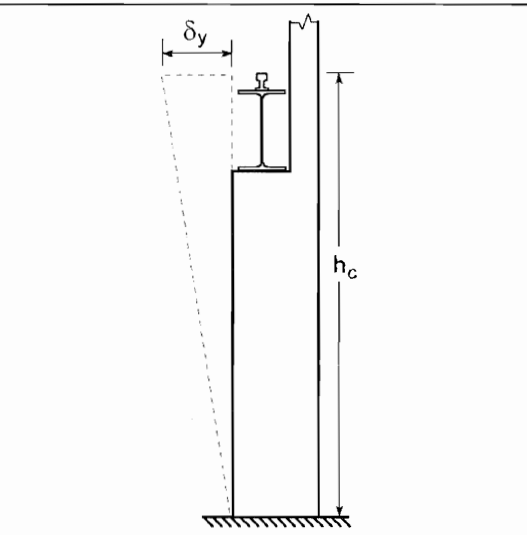
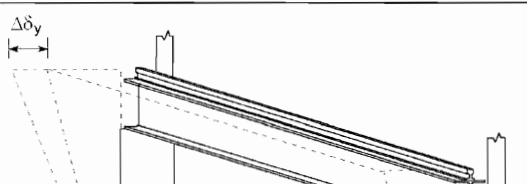
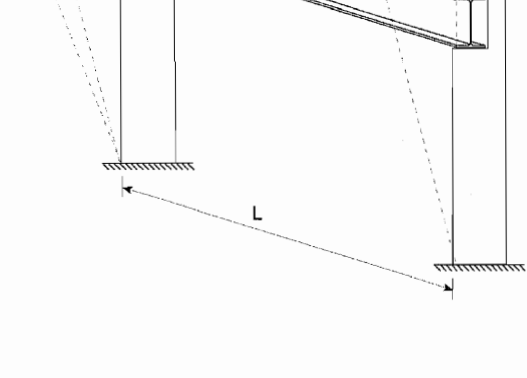
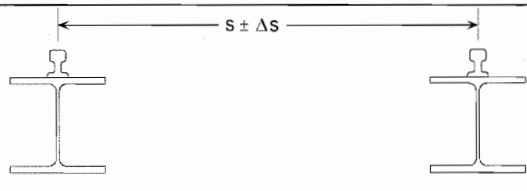
NOTE: Simplified calculation models may be used for stress calculations, provided that the effects of the simplifications are conservative.

7.3 Limits for deformations and displacements

(1) The specific limits, together with the serviceability load combinations under which they apply, should be agreed for each project.

NOTE: The National Annex may specify the limits for vertical and horizontal deflections. The limits given in table 7.1 are recommended for horizontal deflections under the characteristic combination of actions. The limits given in table 7.2 are recommended for vertical deflections under the characteristic combination of actions without any dynamic amplification factors.

Table 7.1: Limiting values of horizontal deflections

Description of deflection (deformation or displacement)	Diagram
<p>a) Horizontal deformation δ_y of a runway beam, measured at the level of the top of the crane rail:</p> $\delta_y \leq L/600$	
<p>b) Horizontal displacement δ_y of a frame (or of a column) at crane support level, due to crane loads:</p> $\delta_y \leq h_c/400$ <p>where: h_c is the height to the level at which the crane is supported (on a rail or on a flange)</p>	
<p>c) Difference $\Delta\delta_y$ between the horizontal displacements of adjacent frames (or columns) supporting the beams of an indoor crane runway:</p> $\Delta\delta_y \leq L/600$	
<p>d) Difference $\Delta\delta_y$ between the horizontal displacements of adjacent columns (or frames) supporting the beams of an outdoor crane runway:</p> <ul style="list-style-type: none"> - due to the combination of lateral crane forces and the in-service wind load: $\Delta\delta_y \leq L/600$ <ul style="list-style-type: none"> - due to the out-of-service wind load $\Delta\delta_y \leq L/400$	
<p>e) Change of spacing Δs between the centres of crane rails, including the effects of thermal changes:</p> $\Delta s \leq 10 \text{ mm} \quad [\text{see Note}]$	
<p>NOTE: Horizontal deflections and deviations of crane runways are considered together in crane design. Acceptable deflections and tolerances depend on the details and clearances in the guidance means. Provided that the clearance c between the crane wheel flanges and the crane rail (or between the alternative guidance means and the crane beam) is also sufficient to accommodate the necessary tolerances, larger deflection limits can be specified for each project if agreed with the crane supplier and the client.</p>	

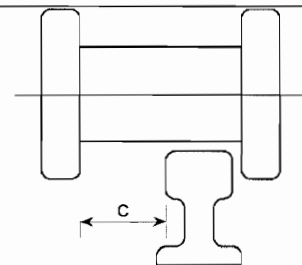
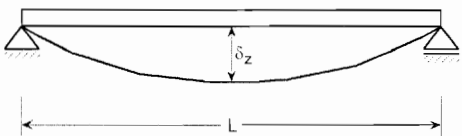
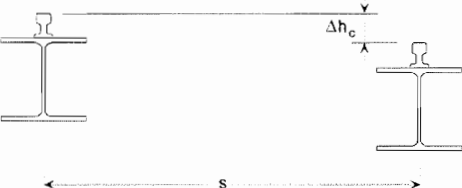
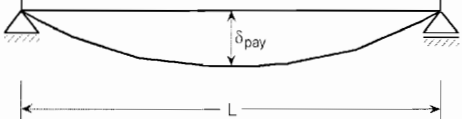


Table 7.2: Limiting values of vertical deflections

Description of deflection (deformation or displacement)	Diagram
<p>a) Vertical deformation δ_z of a runway beam:</p> $\delta_z \leq L/600 \text{ and } \delta_z \leq 25 \text{ mm}$ <p>The vertical deformation δ_z should be taken as the total deformation due to vertical loads, less the possible pre-camber, as for δ_{\max} in figure A1.1 of EN 1990.</p>	
<p>b) Difference Δh_c between the vertical deformations of two beams forming a crane runway:</p> $\Delta h_c \leq s/600$	
<p>c) Vertical deformation δ_{pay} of a runway beam for a monorail hoist block, relative to its supports, due to the payload only:</p> $\delta_{\text{pay}} \leq L/500$	

7.4 Limitation of web breathing

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

(2) Excessive web breathing may be neglected in web panels where the following criterion is satisfied under the frequent load combination, see EN 1990:

$$\sqrt{\left(\frac{\sigma_{x,\text{Ed,ser}}}{k_\sigma \sigma_E}\right)^2 + \left(\frac{1,1 \tau_{\text{Ed,ser}}}{k_\tau \sigma_E}\right)^2} \leq 1,1 \quad (7.1)$$

where: b is the smaller dimension of the web panel;

k_σ, k_τ are the linear elastic buckling coefficients given in EN 1993-1-5;

$$\sigma_E = 190\,000 / (b/t_w)^2 \text{ [N/mm}^2\text{]};$$

$\sigma_{x,\text{Ed,ser}}$ is the direct stress in the web panel;

$\tau_{\text{Ed,ser}}$ is the shear stress in the web panel.

(3) Excessive web breathing may be neglected in web panels without longitudinal stiffeners, in which the ratio b/t_w is less than 120, where t_w is the web thickness.

7.5 Reversible behaviour

(1) To ensure reversible behaviour, the stresses $\sigma_{Ed,ser}$ and $\tau_{Ed,ser}$ resulting from the relevant characteristic load combination or test load combination, calculated making due allowance where relevant for the effects of shear lag in wide flanges and for the secondary effects induced by deformations (for instance secondary moments in trusses) should be limited as follows:

$$\sigma_{Ed,ser} \leq f_y / \gamma_{M,ser} \quad (7.2a)$$

$$\tau_{Ed,ser} \leq \frac{f_y}{\sqrt{3}\gamma_{M,ser}} \quad (7.2b)$$

$$\sqrt{(\sigma_{x,Ed,ser})^2 + 3(\tau_{Ed,ser})^2} \leq f_y / \gamma_{M,ser} \quad (7.2c)$$

$$\sqrt{(\sigma_{x,Ed,ser})^2 + (\sigma_{y,Ed,ser})^2 - (\sigma_{x,Ed,ser})(\sigma_{y,Ed,ser}) + 3(\tau_{Ed,ser})^2} \leq f_y / \gamma_{M,ser} \quad (7.2d)$$

$$\sqrt{(\sigma_{x,Ed,ser})^2 + (\sigma_{z,Ed,ser})^2 - (\sigma_{x,Ed,ser})(\sigma_{z,Ed,ser}) + 3(\tau_{Ed,ser})^2} \leq f_y / \gamma_{M,ser} \quad (7.2e)$$

where: $\sigma_{x,Ed,ser}$ is the direct stress in the longitudinal direction;

$\sigma_{y,Ed,ser}$ is the direct stress in the lateral direction;

$\sigma_{z,Ed,ser}$ is the direct stress in the transverse direction;

$\tau_{Ed,ser}$ is the co-existing shear stress.

NOTE: The numerical value for $\gamma_{M,ser}$ may be defined in the National Annex. The recommended value is 1,00.

(2) The nominal stresses for runway beams supporting top mounted cranes should include the local direct stress $\sigma_{oz,Ed,ser}$ in the web, see 5.7.1, in addition to the global stresses $\sigma_{x,Ed,ser}$ and $\tau_{Ed,ser}$. The bending stress $\sigma_{T,Ed}$ due to the eccentricity of the wheel loads, see 5.7.3, may be neglected.

(3) The nominal stresses for runway beams with a monorail hoist block or an underslung crane should include the local stresses $\sigma_{ox,Ed,ser}$ and $\sigma_{oy,Ed,ser}$ in the bottom flange, see 5.8, in addition to the global stresses $\sigma_{x,Ed,ser}$ and $\tau_{Ed,ser}$.

7.6 Vibration of the bottom flange

(1) The possibility of noticeable lateral vibration of the bottom flange of a simply supported crane runway beam, induced by crane operation or movement, should be avoided.

(2) This may be assumed to be satisfied if the slenderness ratio L/i_z of the bottom flange is not more than 250, where i_z is the radius of gyration of the bottom flange and L is its length between lateral restraints.

8 Fasteners, welds, surge connectors and rails

8.1 Connections using bolts, rivets or pins

(1) See Chapter 3 of EN 1993-1-8.

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

8.2 Welded connections

(1) See Chapter 4 of EN 1993-1-8.

(2) In crane supporting structures, intermittent fillet welds should not be used where they would result in the formation of rust pockets.

NOTE: They can be used where the connection is protected from the weather, e.g. inside box sections.

(3) Intermittent fillet welds should not be used for the web-to-flange connections of runway beams where the welds are subject to local stresses due to the wheel loads.

(4) For high fatigue crane classes, transverse web stiffeners or other attachments should not be welded to the top flanges of runway beams.

NOTE: The National Annex may specify the crane classes to be treated as “high fatigue”. Classes S7 to S9 according to Annex B of EN 1991-3 are recommended.

8.3 Surge connectors

(1) Surge connectors attaching the top flange of a runway beam to the supporting structure should be capable of accommodating:

- the movements generated by the end rotation of the runway beam due to vertical loading, see figure 8.1
- the movements generated by the end rotation of the top flange of the runway beam due to lateral crane forces, see figure 8.2
- the vertical movements associated with the vertical compression of the runway beam and its support, plus wear and settlement of the bearings of the runway beam.

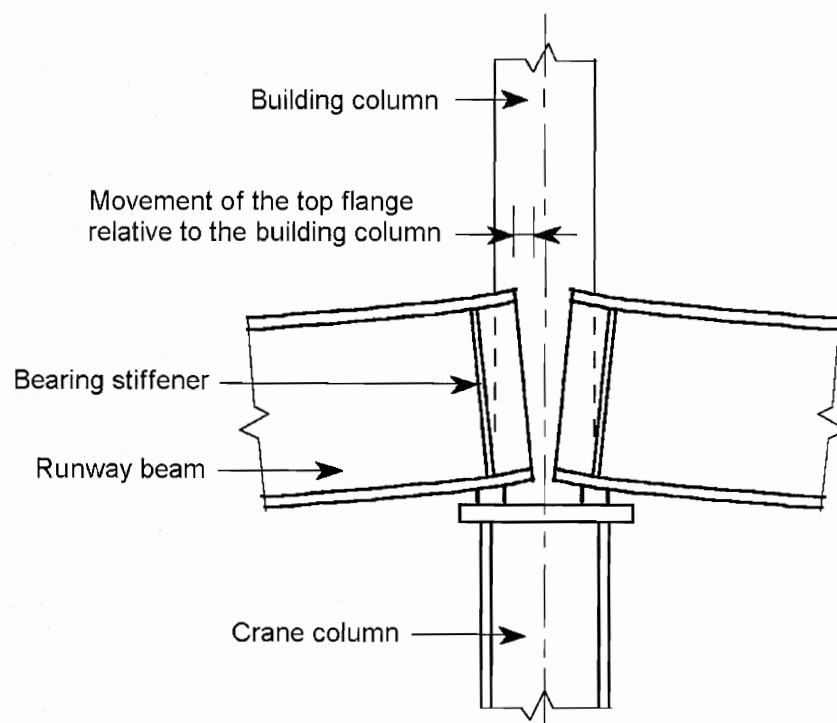


Figure 8.1: End rotation of runway beams

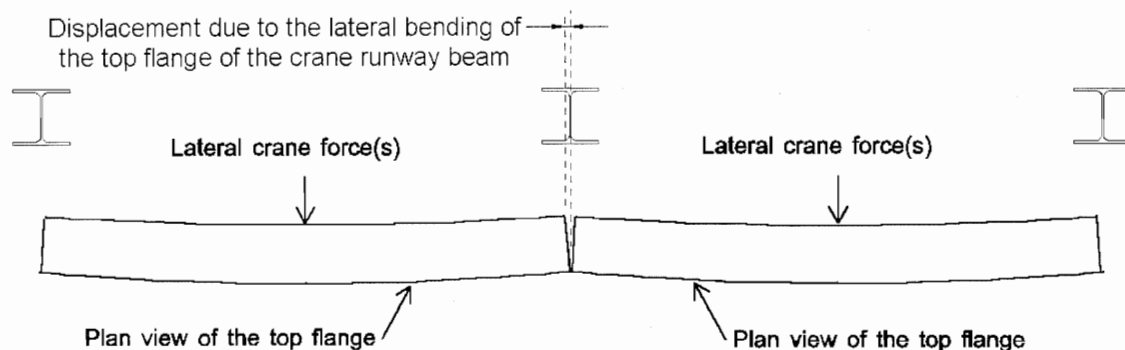


Figure 8.2: End rotation of runway beams due to lateral crane forces

(2) The detailing of the $\overline{\text{AC1}}$ surge connectors $\overline{\text{AC1}}$ and their connections should take into account the possible need for lateral and vertical adjustment of the runway beams in order to maintain the alignment of the crane runway, whilst also respecting the tolerance on location of the rail relative to the centreline of the web of the runway beam.

(3) At supports where no surge connectors are used, the runway beam and the fasteners should be designed to transmit all vertical and horizontal forces from the crane wheels to the support.

8.4 Crane rails

8.4.1 Rail material

(1) The rail steel should comply with 3.6.2.

8.4.2 Design working life

(1) Generally the grade of rail steel should be selected to give the rail an appropriate design working life L_r . Where the design working life of the rail is less than that of the runway beam, see 2.1.3.2, account should be taken of the need for rail replacement in selecting the rail fixings, see 8.5.

8.4.3 Rail selection

(1) The selection of crane rails should take into account the following:

- the rail material;
- the wheel load;
- the wheel material;
- the wheel diameter;
- the crane utilisation.

(2) The contact pressure (Hertz bearing pressure) between crane wheels and rails should be limited to an appropriate value in order:

- to reduce friction;
- to avoid excessive wear of the rail;
- to avoid excessive wear of the wheels.

(3) The method given in $\overline{\text{AC1}}$ ISO 16881-1 $\overline{\text{AC1}}$ should be applied.

8.5 Rail fixings

8.5.1 General

- (1) Depending on their details, crane rail fixings may be classified as rigid or independent.
- (2) Each mechanical rail fixing should normally be designed to resist the maximum lateral horizontal force from one crane wheel. If the wheel spacing is less than the spacing between fixings, their resistance should be increased accordingly.

8.5.2 Rigid fixings

- (1) The following types of crane rail fixings may be classified as rigid:
 - rails welded to runway beams,
 - rails fixed to runway beams by fit bolts, preloaded bolts or rivets that pass through the flange of the rail.
- (2) Crane rails that have rigid rail fixings may be treated as part of the cross-section of the runway beam, provided that due allowance is made for wear of the rail, see 5.6.2(2) and 5.6.2(3).
- (3) Rigid rail fixings should be designed to resist the longitudinal forces developed between the rail and the runway beam plus the lateral forces applied to the rail by the crane wheels.
- (4) Rigid rail fixings should also be checked against fatigue.

8.5.3 Independent fixings

- (1) All crane rail fixings that are not classified as rigid should be classified as independent fixings.
- (2) Independent rail fixings should be designed to resist the lateral forces applied to the rail by the crane wheels.
- (3) A crane rail with independent rail fixings may have suitable elastomeric bearing pads between the rail and the beam.

8.6 Rail joints

- (1) Rails may be either:
 - continuous over the joints of runway beams;
 - discontinuous, with expansion joints.
- (2) In the case of continuous rails, the analysis of the crane supporting structure should be based upon the relevant values of the properties of the rail fixings and bedding for:
 - differential thermal movement;
 - transmission of acceleration and braking forces from the rail to the beam.
- (3) Rail joints should be detailed to minimise impact. As a minimum, a bevel joint offset from the ends of the runway beams (see figure 8.3) should be used.

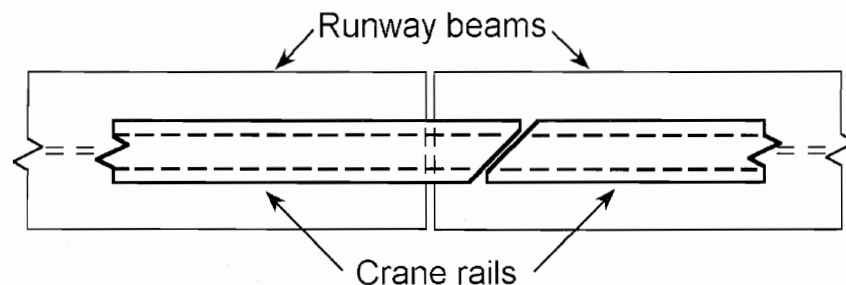


Figure 8.3: Offset bevel joint in crane rail

9 Fatigue assessment

9.1 Requirement for fatigue assessment

- (1) Fatigue assessments according to EN 1993-1-9 should be carried out for all critical locations.
- (2) Fatigue assessment need not be carried out for crane supporting structures if the number of cycles at more than 50% of full payload does not exceed C_0 .

NOTE: The numerical value for C_0 may be defined in the National Annex. The recommended value is 10^4 .

- (3) A fatigue assessment is generally required only for those components of the crane supporting structure that are subject to stress variations from vertical crane loads.

NOTE: Stress variations from horizontal crane loads are normally negligible. However, in some cases surge connections can suffer fatigue due to lateral crane loads. Also, for some types of crane supporting structures and crane operations, fatigue can result from multiple acceleration and braking actions.

- (4) For members that might be subject to wind-induced vibrations, see EN 1991-1-4.

9.2 Partial factors for fatigue

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

NOTE: The numerical value for γ_{Ff} may be defined in the National Annex. The recommended value is 1.0.

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

NOTE: The National Annex may define the values for γ_{Mf} . The use of table 3.1 in EN 1993-1-9 is recommended.

9.3 Fatigue stress spectra

9.3.1 General

- (1) The stresses σ_p and τ_p taken into account in fatigue verifications should be the nominal stresses (including both global and local effects) determined using elastic analysis.
- (2) Where full information on the details of crane operation and data on the cranes are all available during design, fatigue stress histories from crane operations should be determined for each critical detail, using Annex A of EN 1993-1-9.
- (3) Where such information does not exist or where a simplified approach needs to be used, the fatigue loads from crane operations may be taken from 2.12.1(4) of EN 1991-3.
- (4) The secondary moments due to joint rigidity and chord member continuity in members of lattice girders, lattice surge girders and triangulated bracing panels should be included as specified in 5.9.

9.3.2 Simplified approach

- (1) For the simplified fatigue loading specified in 2.12.1(4) of EN 1991-3, the following procedure may be used to determine the design stress range spectrum.

NOTE: The simplified fatigue load $\overline{AC1} Q_e = \varphi_{fat} \lambda_i Q_{max,i} \overline{AC1}$ from EN 1991-3 is already related to 2×10^6 cycles.

- (2) The maximum stresses $\sigma_{p,max}$ and $\tau_{p,max}$ and the minimum stresses $\sigma_{p,min}$ and $\tau_{p,min}$ due to the simplified fatigue load Q_e should be determined for the relevant detail.
- (3) The damage equivalent stress range related to $\overline{AC1} 2 \times 10^6 \overline{AC1}$ cycles $\Delta\sigma_{E2}$ and $\Delta\tau_{E2}$ may be obtained from:

$$\Delta\sigma_{E2} = |\sigma_{p,max} - \sigma_{p,min}| \quad (9.1)$$

$$\Delta\tau_{E2} = |\tau_{p,max} - \tau_{p,min}| \quad (9.2)$$

- (4) Where the number of stress cycles is higher than the number of crane working cycles, see figure 9.1, the equivalent load Q_e according to 2.12.1(4) of EN 1991-3 should be determined using this higher number as the total number of working cycles C in table 2.11 of EN 1991-3.

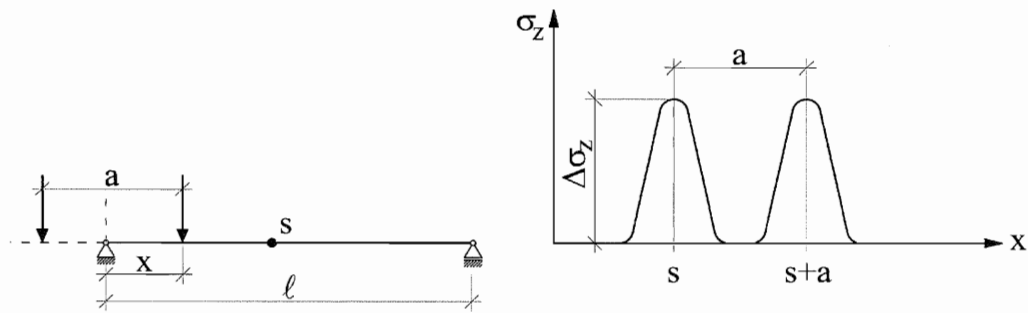


Figure 9.1: Example of two stress cycles arising from one crane working cycle

9.3.3 Local stresses due to wheel loads on the top flange

(1) In the web, the following local stresses due to wheel loads on the top flange should be taken into account:

- compressive stresses $\sigma_{z,Ed}$ as specified in 5.7.1, [AC1] without assuming contact between flange and web in case of not fully penetrated welds [AC1]
- shear stresses $\tau_{xz,Ed}$ as specified in 5.7.2,
- unless specified otherwise, bending stresses $\sigma_{T,Ed}$ due to the lateral eccentricity e_y of vertical loads $F_{z,Ed}$ as specified in 5.7.3.

NOTE: The National Annex may define crane classes for which the bending stresses $\sigma_{T,Ed}$ can be neglected. Crane classes S_0 to S_3 are recommended.

[AC1] (2) For partial penetration and fillet welds the compressive and shear stresses calculated for the web thickness should be transformed to the stresses of the weld. See Table 8.10 in EN 1993-1-9 [AC1]

(3) If the rail is welded to the flange, the local stresses in the welds connecting the rail to the flange should be taken into account [AC1] without assuming contact between flange and rail [AC1].

9.3.4 Local stresses due to underslung trolleys

(1) The local bending stresses in the bottom flange due to wheel loads from underslung trolleys, see 5.8, should be taken into account.

9.4 Fatigue assessment

9.4.1 General

(1) See section 8 of EN 1993-1-9.

9.4.2 Multiple crane actions

(1) For a member loaded by two or more cranes, the total damage should satisfy the criterion:

$$\sum_i D_i + D_{\text{dup}} \leq 1 \quad (9.3)$$

where: D_i is the damage due to a single crane i acting independently;

D_{dup} is the additional damage due to combinations of two or more cranes occasionally acting together.

(2) The damage D_i due to a single crane i acting independently should be calculated from the direct stress range or the shear stress range or both, depending upon the constructional detail, see EN 1993-1-9, using:

$$D_i = \left[\frac{\gamma_{Ff} \Delta \sigma_{E2,i}}{\Delta \sigma_c / \gamma_{Mf}} \right]^3 + \left[\frac{\gamma_{Ff} \Delta \tau_{E2,i}}{\Delta \tau_c / \gamma_{Mf}} \right]^5 \quad (9.4)$$

where: $\Delta \sigma_{E2,i}$ is the equivalent constant amplitude direct stress range for a single crane i ;

$\Delta \tau_{E2,i}$ is the equivalent constant amplitude shear stress range for a single crane i .

(3) The additional damage D_{dup} due to two or more cranes occasionally acting together should be calculated from the direct stress range or the shear stress range or both, depending on the constructional detail, see EN 1993-1-9, using:

$$D_{dup} = \left[\frac{\gamma_{Ff} \Delta \sigma_{E2,dup}}{\Delta \sigma_c / \gamma_{Mf}} \right]^3 + \left[\frac{\gamma_{Ff} \Delta \tau_{E2,dup}}{\Delta \tau_c / \gamma_{Mf}} \right]^5 \quad (9.5)$$

where: $\Delta \sigma_{E2,dup}$ is the equivalent constant amplitude direct stress range due to two or more cranes acting together;

$\Delta \tau_{E2,dup}$ is the equivalent constant amplitude shear stress range due to two or more cranes acting together.

(4) If two cranes are intended to act together (in tandem or otherwise) to a substantial extent, the two cranes should be treated as comprising one single crane.

(5) In the absence of better information, the equivalent constant amplitude stress range $\Delta \sigma_{E2}$ due to two or more cranes occasionally acting together may be obtained by applying damage equivalence factors λ_{dup} .

NOTE: The National Annex may define the values of the factors λ_{dup} . It is recommended to take a value of λ_{dup} equal to the values λ_i from table 2.12 of EN 1991-3 for a loading class S_i as follows:

- for 2 cranes: 2 classes below the loading class of the crane with the lower loading class;
- for 3 or more cranes: 3 classes below the loading class of the crane with the lowest loading class.

9.5 Fatigue strength

(1) See tables 8.1 to 8.10 of EN 1993-1-9.

Annex A [informative] – Alternative assessment method for lateral-torsional buckling

NOTE: Where specified in the National Annex, the method given in this Annex A may be used as an alternative to the method given in 6.3.2.3(1).

A.1 General

(1) This method may be used to check the lateral-torsional buckling resistance of a simply supported runway beam of uniform cross-section, with vertical actions and lateral horizontal actions applied eccentrically relative to its shear centre.

(2) The actions should be expressed as vertical and horizontal forces applied through the shear centre, together with a warping torsional moment T_w .

A.2 Interaction formula

(1) Members that are subjected to combined bending and torsion should satisfy:

$$\boxed{\text{AC1}} \frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk} / \gamma_{M1}} + \frac{C_{mz} M_{z,Ed}}{M_{z,Rk} / \gamma_{M1}} + \frac{k_w k_{zw} k_\alpha B_{Ed}}{B_{Rk} / \gamma_{M1}} \leq 1 \quad \boxed{\text{AC1}} \quad (\text{A.1})$$

where: C_{mz} is the equivalent uniform moment factor for bending about the z-z axis, according to EN 1993-1-1 Table B.3;

$$\boxed{\text{AC1}} k_w = 0,7 - \frac{0,2 B_{Ed}}{B_{Rk} / \gamma_{M1}} \quad \boxed{\text{AC1}}$$

$$k_{zw} = 1 - \frac{M_{z,Ed}}{M_{z,Rk} / \gamma_{M1}}$$

$$k_\alpha = \frac{1}{1 - M_{y,Ed} / M_{y,cr}}$$

$M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the maximum moments about the y-y and z-z axis respectively;

$M_{y,Rk}$ and $M_{z,Rk}$ are the characteristic values of the resistance moment of the cross-section about its y-y and z-z axis respectively, from EN 1993-1-1 Table 6.7;

$M_{y,cr}$ is the elastic critical lateral-torsional buckling moment about the y-y axis;

$\boxed{\text{AC1}} B_{Ed} \quad \boxed{\text{AC1}}$ is the design value of the warping torsional moment;

$\boxed{\text{AC1}} B_{Rk} \quad \boxed{\text{AC1}}$ is the characteristic value of the warping torsional resistance moment;

χ_{LT} is the reduction factor for lateral-torsional buckling according to 6.3.2 of EN 1993-1-1.

(2) The reduction factor χ_{LT} may be determined from 6.3.2.3 of EN 1993-1-1 for rolled or equivalent welded sections with equal flanges, or with unequal flanges, taking b as the width of the compression flange, provided that:

$$I_{z,t} / I_{z,c} \geq 0,2$$

where: $I_{z,c}$ and $I_{z,t}$ are the second moments of area about the z-z axis for the compression and tension flanges respectively.