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Eurocode 3 : Design of steel structures

Part 1-1 : General structural rules

Calcul des structures en acier

Partie 1-1 : Règles générales

Bemessung und Konstruktion von Stahlbauten

Teil 1-1 : Allgemeine Bemessungsregeln

CEN

European Committee for Standardisation
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Content

Foreword

This European Standard EN 1993-1-1, Design of Steel Structures : General rules, has been prepared on behalf of Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1993-1-1 on YYYY-MM-DD.

No existing European Standard is superseded.

Background of the Eurocode programme

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

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

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

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

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

EN 1990	Eurocode:	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.

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

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

National Standards implementing Eurocodes

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

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

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

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

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

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

[3] According to Art. 12 of the CPD the interpretative documents shall :
a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;
c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.
The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

[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.

Additional information specific to EN 1993-1

EN 1993 is intended to be used with Eurocodes EN 1990 – Basis of Structural Design, EN 1991 – Actions on structures and EN 1992 to EN 1999, when steel structures or steel components are referred to.

EN 1993-1 is the first part of seven parts of EN 1993 – Design of Steel Structures – and gives generic design rules that are intended to be used with the other parts EN 1993-2 to EN 1993-7.

EN 1993-1 comprises eleven subparts EN 1993-1-1 to EN 1993-1-11 each addressing specific steel components, limit states or materials.

It may also be used for design cases not covered by the Eurocodes (other strucutres, other actions, other materials) serving as a reference document for other CEN TC´s concerning structural matters.

EN 1993-1 is intended for use by

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

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

National annex for EN 1993-1

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-1 through clauses:

- 2.3.1(1)
- 3.1(3)
- 3.2.1(1)
- 3.2.3(1)
- 5.4.2.1(3)
- 6.1(1)
- 6.3.4(4)
- 6.3.4(6)
- A.1(3)

1 General

1.1 Scope

1.1.1 Scope of Eurocode 3

(1) Eurocode 3 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 design.

(2) Eurocode 3 is only concerned with requirements for resistance, serviceability, durability and fire resistance of steel structures. Other requirements, e.g concerning thermal or sound insulation, are not considered.

(3) Eurocode 3 is intended to be used in conjunction with:

- EN 1990 “Basis of structural design”
- EN 1991 “Actions on structures”
- EN´s for construction products relevant for steel structures
- EN xxx[5] “Requirements for the execution of steel structures”
- EN 1992 to EN 1999 when steel structures or steel components are referred to

(4) Eurocode 3 is subdivided in various parts:

EN 1993-1 Design of Steel Structures : Generic rules.

EN 1993-2 Design of Steel Structures : Steel bridges.

EN 1993-3 Design of Steel Structures : Buildings.

EN 1993-4 Design of Steel Structures : Silos, tanks and pipelines.

EN 1993-5 Design of Steel Structures : Piling.

EN 1993-6 Design of Steel Structures : Crane supporting structures.

EN 1993-7 Design of Steel Structures : Towers, masts and chimneys.

(5) The parts EN 1993-2 to EN 1993-7 refer to the Generic rules in Part 1. the clauses in parts EN 1993-2 to EN 1993-7 supplement the clauses in EN 1993-1.

(6) EN 1993-1 “Generic rules” comprises:

EN 1993-1-1 Design of Steel Structures : General structural rules.

EN 1993-1-2 Design of Steel Structures : Structural fire design.

EN 1993-1-3 Design of Steel Structures : Cold-formed thin gauge members and sheeting.

EN 1993-1-4 Design of Steel Structures : Stainless steels.

EN 1993-1-5 Design of Steel Structures : Plated structural elements.

EN 1993-1-6 Design of Steel Structures : Strength and stability of shell structures.

EN 1993-1-7 Design of Steel Structures : Strength and stability of planar plated structures transversely loaded.

EN 1993-1-8 Design of Steel Structures : Design of joints.

EN 1993-1-9 Design of Steel Structures : Fatigue strength of steel structures.

EN 1993-1-10 Design of Steel Structures : Selection of materials for fracture toughness and through thickness properties.

[5] EN xxx is the conversion of ENV 1090

EN 1993-1-11 Design of Steel Structures : Use of high strength tensile elements.

1.1.2 Scope of Part 1.1 of Eurocode 3

(1) The Part 1-1 of Eurocode 3 gives basic design rules for steel structures.

(2) The following subjects are dealt with in Part 1-1:

Section 1: Introduction

Section 2: Basis of design

Section 3: Materials

Section 4: Durability

Section 5: Structural analysis

Section 6: Ultimate limit states

Section 7: Serviceability limit states

Section 8: Fasteners, weld, connections and joints

(3) Section 1 to 2 provide additional clauses to those given in EN 1990 "Basis of structural design".

(4) Section 3 deals with material properties of products made of low allow structural steels.

(5) Section 4 gives general rules for durability.

(6) Section 5 refers to the structural analysis of bar structures.

(7) Section 6 gives detailed rules for the design of cross sections and members.

(8) Section 7 given rules for serviceability.

1.2 Normative references

(1) The following normative documents contain provisions which, through references in this text, constitute provisions of this European standard. For dated references, subsequent amendments to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

1.2.1 General reference standards

EN 10020:2000 Definition and classification of grades of steels.

EN 10021:1993 General technical delivery requirements for steel and iron products.

EN 10079:1992 Definitions of steel products

EN xxx[5] Requirements for the execution of steel structures

1.2.2 Weldable structural steel reference standards

EN 10025-1: September 2000 Hot-rolled products of structural steels - Part 1: General delivery conditions.

EN 10025-2: September 2000 Hot-rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels.

EN 10025-3: September 2000 Hot-rolled products of structural steels - Part 3: Technical delivery conditions for normalized / normalized rolled weldable fine grain structural steels.

EN 10025-4: September 2000 Hot-rolled products of structural steels - Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels.

EN 10025-5: September 2000 Hot-rolled products of structural steels - Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance.

EN 10025-6: September 2000 Hot-rolled products of structural steels - Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition.

EN 10163:1991 Delivery requirements for surface condition of hot-rolled steel plates, wide flats and sections - Part 1: General requirements - Part 2: plates and wide flats - Part 3: Sections.

EN 10164:1993 Steel products with improved deformation properties perpendicular to the surface of the product - Technical delivery conditions.

EN 10210-1:1994 Hot finished structural hollow sections of non-alloy and fine grain structural steels – Part 1: Technical delivery requirements.

EN 10219-1:1997 Cold formed hollow sections of structural steel - Part 1: Technical delivery requirements.

1.2.3 Dimensions of sections reference standards

EN 10024:1995 Hot rolled taper flange I sections – Tolerances on shape and dimensions.

EN 10029:1991 Hot rolled steel plates 3 mm thick or above - Tolerances on dimensions, shape and mass.

EN 10034:1993 Structural steel I- and H- sections - Tolerances on shape and dimensions.

EN 10048:1996 Hot rolled narrow teel strip – Tolerances on shape and dimensions.

EN 10051:1997 Continuously hot-rolled uncoated plate, sheet and strip of non-alloy and alloy steels – Tolerances on dimensions and shape.

EN 10055:1995 Hot rolled steel equal flange tees with radiused root and toes - Dimensions and tolerances on shape and dimensions.

EN 10056-1:1998 Structural steel equal and unequal leg angles - Part 1: Dimensions.

EN 10056-2:1993 Structural steel equal and unequal leg angles - Part 2: Tolerances on shape and dimensions.

EN 10067:1996 Hot rolled bulb flats – Dimensions and tolerances on shape, dimensions and mass.

EN 10210-2:1997 Hot finished structural hollow sections of non-alloy and fine grain structural steels – Part 2: Tolerances, dimensions and sectional properties.

EN 10219-2:1997 Cold formed hollow sections of structural steel - Part 2: Dimensions, tolerances and geometrical properties.

EN 10279:2000 Hot rolled steel channel- Tolerances on shape and dimensions.

1.3 Assumptions

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

– fabrication and erection complies with EN xxx[5]

1.4 Distinction between principles and application rules

(1) The rules in EN 1990 clause 1.4 apply.

1.5 Definitions

(1) The rules in EN 1990 clause 1.5 apply.

(2) The following terms are used in Part 1-1 of Eurocode 3 with the following meanings:

1.5.1
frame
total or portion of a structure, comprising an assembly of directly connected structural elements, designed to act together to resist load; this term refers to both moment frames and triangulated frames; it covers both plane frames and three-dimensional frames

1.5.2
sub-frame
a frame which forms part of a larger frame, but may be treated as an isolated frame in a structural analysis

1.5.3
type of framing
terms used to distinguish between frames which are either:

- **semi-continuous**, in which the structural properties of the members and connections need explicit consideration in the global analysis
- **continuous**, in which only the structural properties of the members need be considered in the global analysis
- **simple**, in which the joints are not required to resist moments

1.5.4
global analysis
the determination of a consistent set of internal forces and moments in a structure, which are in equilibrium with a particular set of actions on the structure

1.5.5
system length
distance between two adjacent points at which a member is braced against lateral displacement in a given plane, or between one such point and the end of the member

1.5.6
buckling length
system length of an otherwise similar member with pinned ends, which has the same buckling resistance as a given member

1.5.7
shear lag effect
non-linear stress distribution in wide flanges due to shear deformations that is taken into account by a reduction of the flange width in safety assessment

1.6 Symbols

(1) For the purpose of this standard the following symbols apply.

Draft note: ... to be inserted later.

1.7 Conventions for member axes

(1) In general the convention for member axes is:

x-x - along the member

y-y - axis of the cross-section

z-z - axis of the cross-section

(2) For steel members, the conventions used for cross-section axes are:

– generally:

y-y - cross-section axis parallel to the flanges

z-z - cross-section axis perpendicular to the flanges

– for angle sections:

y-y - axis parallel to the smaller leg

z-z - axis perpendicular to the smaller leg

– where necessary:

u-u - major axis (where this does not coincide with the yy axis)

v-v - minor axis (where this does not coincide with the zz axis)

(3) The symbols used for dimensions and axes of rolled steel sections are indicated in Figure 1.1.

(4) The convention used for subscripts which indicate axes for moments is: "Use the axis about which the moment acts."

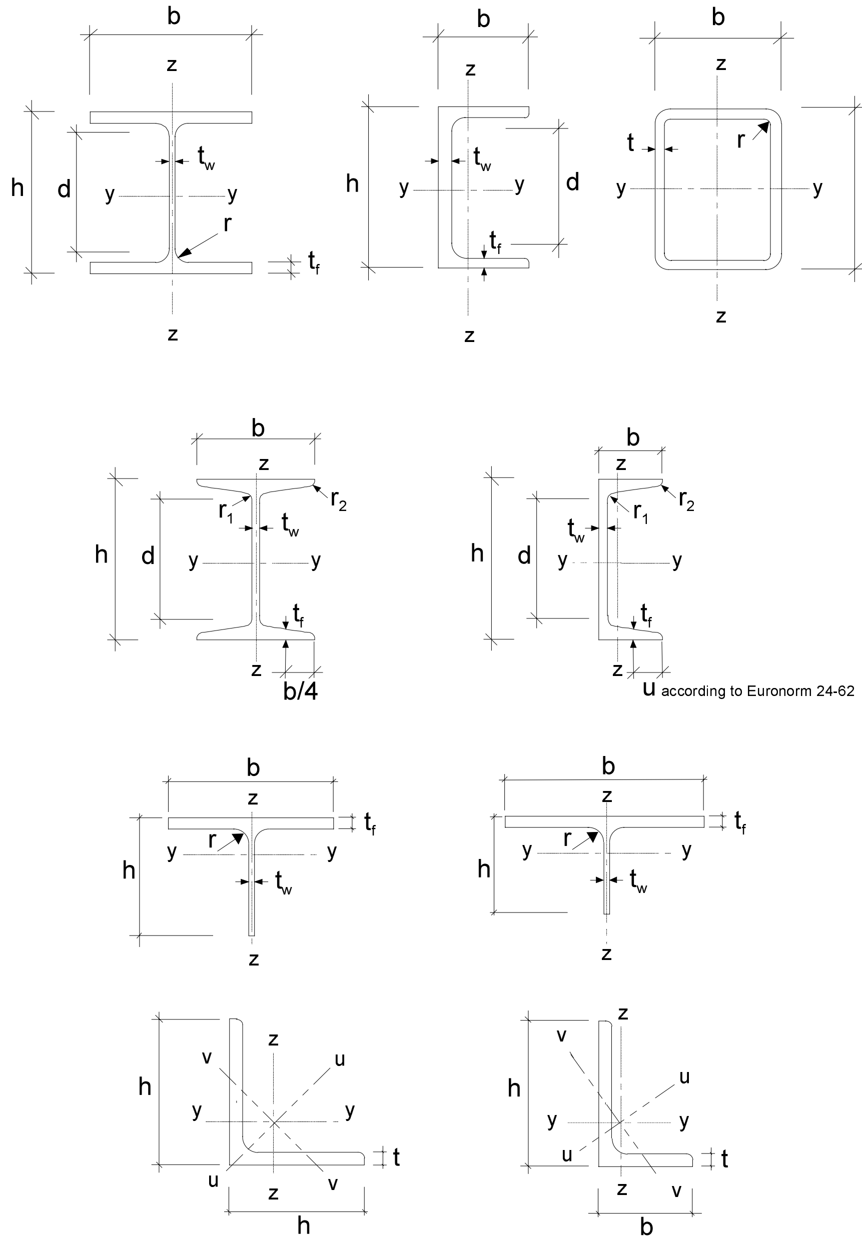


Figure 1.1: Dimensions and axes of sections

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1) The design of steel structures shall be in accordance with the general rules given in EN 1990.

(2) The supplementary provisions for steel structures given in this section shall also be applied.

(3) The basic requirements of EN 1990 section 2 are deemed be satisfied for steel structures when limit state design in conjunction with the partial factor method using EN 1990 and EN 1991 for actions and their combinations and EN 1993 for resistances, rules for serviceability and durability are applied.

2.1.2 Reliability management

(1) When different levels of reliability are required, these levels should be preferably achieved by an appropriate choice of quality management in design and execution, according to EN 1990 Annex C and EN xxx[5].

2.1.3 Design working life, durability and robustness

(1) Depending on the type of action affecting durability and the design working life (see EN 1990) steel structures should be

- designed for corrosion with
  - suitable surface protection (see EN ...)
  - use of weathering steel
  - use of stainless steel (see Part 1-4)
- detailed for sufficient fatigue life (see Part 1-9)
- designed for wearing
- inspected and maintained.

2.2 Principles of limit state design

(1) The resistances of cross sections and members specified in this Eurocode 3 for the ultimate limit states as defined in EN 1990-3.3 are based on tests in which the material exhibited sufficient ductility to apply simplified design models. The simplifications comprise

- the use of nominal stresses without considering notch effects,
- the neglection of residual stresses from fabrication and erection in tension elements,
- the neglection of constraints due to non linear temperature distribution,
- the adoption of stress bloc distributions for plastic resistances,
- the neglection of secondary bending moments from deformations in joints of trusses,
- the assumptions of uniform stress distributions in fillet welds, see Part 1-8,
- the assumption of uniform force distributions in groups of fasteners, see Part 1-8,
- the neglection of effects of tolerances in joints.

(2) The resistances specified in this Eurocode Part may therefore only be used when the conditions for materials in section 3 are met and when the choice of material complies with the conditions in Part 1-10 to avoid brittle fracture.

2.3 Basic variables

2.3.1 Actions and environmental influences

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

NOTE The National Annex may define actions for particular regional or climatic or accidental situations.

(2) The actions to be considered in the erection stage should be obtained from EN 1993-1-6.

(3) The effects of predicted absolute and differential settlements should be considered as best estimetes of imposed deformations.

(4) The effects of uneven settlements or imposed deformations or other forms of prestressing imposed during erection should be considered by their nominal value P_k as permanent action and grouped with other permanent actions G_k to a singular action ($G_k + P_k$).

(5) For structures for which the formation of plastic mechanisms may be taken into account and 2^{nd} order effects do not apply indirect actions caused by uneven settlements or imposed deformations need not be considered in ultimate limit states.

(6) Fatigue actions not defined in EN 1991 should be determined according to Annex A of Part 1-9.

2.3.2 Material and product properties

(1) Material properties for steels and construction products and geometrical data to be used for design should be those specified in the relevant hEN´s or ETA´s unless otherwise indicated in this standard.

2.4 Verification by the partial factor method

2.4.1 Design value of material property

(1) For the design of steel structures characteristic value X_K or nominal values X_n of material property shall be used as indicated in this Eurocode.

2.4.2 Design value of geometrical data

(1) Geometrical data for cross sections and systems may be taken as nominal values from product standards hEN or drawings for the execution to EN xxx[5].

(2) Design values of geometrical imperfections specified in this standard comprise

- the effects of geometrical imperfections of members as controlled by geometrical tolerances in product standards or the execution standard.
- the effects of structural imperfections from fabrication and erection, residual stresses, distribution of yield strength.

2.4.3 Design resistances

(1) For steel structures equation (6.6c) or equation (6.6d) of EN 1990 applies:

$$R_d = \frac{R_k}{\gamma_M} = \frac{1}{\gamma_M} R_k \left(\eta_1 X_{k1}; \eta_i X_{ki}; a_d\right) \qquad (2.1)$$

where R_k is the characteristic value of resistance of a cross section or member determined with characteristic or nominal values for the material properties

γ_M is the global partial factor for the particular resistance

2.4.4 Verification of static equilibrium (EQU)

(1) The reliability formate for the verification of static equilibrium in Table 1.2 (A) in Annex A of EN 1990 also applies to design situations equivalent to (EQU), e.g. for the design of hold down anchors or the verification of up lift of bearings of continuous beams.

2.5 Design assisted by testing

(1) The resistances R_K in this standard have been determined according to Annex D of EN 1990.

(2) In recommending classes of constant partial factors γ_{Mi} the characteristic values R_K were obtained from

$$R_K = R_d \cdot \gamma_{Mi} \tag{2.2}$$

where R_d are design values according to Annex D of EN 1990

γ_{Mi} are recommended partial factors.

NOTE 1 The numerical values of the recommended partial factors γ_{Mi} have been determined such that R_K represents approximately the 5 %-fractile for an infinite number of tests.

NOTE 2 For characteristic values of fatigue strength and partial factors γ_{Mf} for fatigue see Part 1-9.

NOTE 3 For characteristic values of toughness resistance and safety elements for the toughness verification see Part 1-10.

(3) When resistances R_K for prefabricated products shall be determined from tests, the procedure in (2) should be considered.

3 Materials

3.1 General

(1) The nominal values of material properties given in this section shall be adopted as characteristic values in design calculations.

(2) This Part of EN 1993 covers the design of steel structures fabricated from steel material conforming to the steel grades listed in Table 3.1.

(3) Other materials and products may only be used if their use is evaluated in accordance with the relevant rules in this standard and the applicable National Annex.

3.2 Structural steel

3.2.1 Material properties

(1) The nominal values of the yield strength f_y and the ultimate strength f_u for structural steel shall be obtained

a) either in direct use of the values $f_y = R_{eh}$ and $f_u = R_m$ of the product standard

b) or by using the simplification given in Table 3.1

NOTE For the choice of the method see National Annex.

3.2.2 Ductility requirements

(1) The design rules are based on the following requirements:

- unless otherwise specified the ratio of the specified minimum ultimate tensile strength f_u to the specified minimum yield strength f_y should satisfy:

  $f_u / f_y \geq 1{,}10$

- the elongation at failure on a gauge length of $5{,}65\sqrt{A_o}$ (where A_0 is the original cross-section area) is not less than 15%;
- the ultimate strain ε_u is at least 15 times the yield strain ε_y , where ε_u corresponds to the ultimate strength f_u .

(2) The steel grades listed in Table 3.1 are accepted as satisfying these requirements.

3.2.3 Fracture toughness

(1) The material shall have sufficient fracture toughness to avoid brittle fracture of tension elements at the lowest service temperature expected to occur within the intended design life of the structure.

NOTE The lowest service temperature to be adopted in design may be given in the National Annex.

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

Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Thickness t mm *)			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
EN 10025				
S 235	235	360	215	340
S 275	275	430	255	410
S 355	355	510	335	490
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	540	390	520
S 460 N/NL	460	570	430	550
S 275 M/ML	275	380	255[1)]	360[1)]
S 355 M/ML	355	470	335[1)]	450[1)]
S 420 M/ML	420	520	390[1)]	500[1)]
S 460 M/ML	460	550	430[1)]	530[1)]
S 460 Q/QL/QL1	460	570	440	550
S 235 W	235	360	215	340
S 355 W	355	510	335	490

1) For flat products: $40 \text{ mm} < t \leq 63$ mm only;
*) t is the nominal thickness of the element

Table 3.1 (continued): Nominal values of yield strength f_y and ultimate tensile strength f_u for structural hollow sections

Standard and steel grade	Thickness t mm *)			
	$t \leq 40$ mm		40 mm $< t \leq 65$ mm	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
EN 10210				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 460 NH/NLH	460	560	430	550
EN 10219				
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S 420 MH/MLH	420	500		
S 460 MH/MLH	460	530		
*) t is the nominal thickness of the element				

3.2.4 Through thickness properties

(1) Steel with improved through thickness properties to EN 10164 should be used where necessary.

NOTE Guidance on the choice of through thickness properties is given in prEN 1993-1-10.

3.2.5 Tolerances

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

(2) For structural analysis and design the nominal values of dimensions should be used.

3.2.6 Design values of material coefficients

(1) The material coefficients to be adopted in calculations for the structural steels covered by this Eurocode Part should be taken as follows:

- modulus of elasticity $E = 210000\,\mathrm{N/mm^2}$
- shear modulus $G = E/2(1+\nu)$
- Poisson's ratio $\nu = 0{,}3$

– coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ per°C

– unit weight $\gamma = 78{,}5$ kN/m^3

(2) For calculating the stress effects of unequal temperatures in composite concrete-steel structures the coefficient of linear thermal expansion may be taken as $\alpha = 10 \times 10^{-6}$ per°C .

3.3 Connecting devices

3.3.1 Fasteners

(1) Requirements for fasteners are given in EN 1993-1-8.

3.3.2 Welding consumables

(1) Requirements for welding consumables are given in EN 1993-1-8.

4 Durability

(1) The basic requirements for durability are set out in EN 1990.

(2) The means of executing the protective treatment undertaken off-site and on-site shall be in accordance with EN xxx[5].

NOTE EN xxx[5] lists the factors affecting execution that need to be specified during design.

(3) Parts susceptible to corrosion, mechanical wear or fatigue should be designed such that inspection, maintenance and reconstruction can be achieved in a way suitable to the design life and access available for in-service inspection and maintenance.

NOTE Adopting a suitable inspection program can considerably extend durability.

5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

(1) For the verification of the basic design requirements for the ultimate limit state, the serviceability limit state and for fatigue as given in EN 1990 the action effects shall be calculated on the basis of a modelling of the steel structure and its components that is appropriate to the limit state under consideration.

(2) The calculation model and basic assumptions for the calculations shall reflect the structural behaviour in the ultimate limit state and serviceability limit state with appropriate accuracy and reflect the anticipated type of behaviour of the cross-sections, members, joints and bearings.

(3) The design assumptions shall be consistent with the method used for the analysis.

5.1.2 Joint modelling

(1) The effects of the behaviour of the joints on the distribution of internal forces and moments within a structure, and on the overall deformations of the structure, should generally taken into account, but where these effects are sufficiently small they may be neglected.

(2) To identify whether the effects of joint behaviour on the analysis need be taken into account, a distinction may be made between three joint models as follows, see EN 1993-1-8, 5.1.1:

- simple, in which the joint may be assumed not to transmit bending moments;
- continuous, in which the behaviour of the joint may be assumed to have no effect on the analysis;
- semi-continuous, in which the behaviour of the joint needs to be taken into account in the analysis

(3) The requirements of the various types of joints are given in prEN 1993-1-8.

(4) Where semi-continuous joints are used, the initial value of the rotational stiffness (as specified in EN 1993-1-8) should be used when calculating elastic critical loads or buckling lengths.

5.1.3 Ground structure interaction

(1)P Account shall be taken of the deformation characteristics of the supports where relevant.

(2) If the distribution of internal forces and moments in the structure is not significantly altered by ground structure interactions like rotations or settlements, see also EN 1997, the effects of ground structure interaction on the structure may be disregarded.

5.2 Structural stability

5.2.1 Effects of deformed geometry of the structure

(1) The internal forces and moments may generally be determined using either:

- first-order theory, using the initial geometry of the structure.
- second-order theory, taking into account the influence of the deformation of the structure.

(2) The first-order theory may be used for the global analysis, if the increase of the relevant internal forces and moments caused by the deformations according to first order theory is less than 10%. This condition is fulfilled, if the following criteria applies:

$$\alpha_{cr} = \frac{N_{cr}}{N_{Ed}} \geq 10 \qquad (5.1)$$

where α_{cr} is the ratio of the elastic critical force for the relevant buckling mode to the design value of the compression force

5.2.2 Methods of analysis

(1) If according to 5.2.1 the influence of the deformation of the structure has to be taken into account (2) or (3) should be applied to consider these effects and to verify the structural stability.

(2) The stability of structures or their parts in general is accounted for by second order-analysis of the structure taking account of imperfections, see 5.3, and following one of the the two methods a) or b):

a) if the imperfections and second order effects of the individual members (see 5.3.4) are included in the second order analysis of the structure, no individual stability check for the members according to 6.3 is necessary.

b) if the imperfections and second order effects of the individual members are not included in the second order analysis of the structure, the individual stability of the members shall be checked according to 6.3. This verification should take account of end moments and forces from the second order analysis of the system including e.g. the horizontal sway according to 5.3.2(6) and should be based on a buckling length equal to the system length.

5.3 Imperfections

5.3.1 Basis

(1) Appropriate allowances shall be incorporated to cover the effects of imperfections, including residual stresses and geometrical imperfections such as lack of verticality, lack of straightness, lack of flatness, lack of fit and the unavoidable minor eccentricities present in connections of the unloaded structure.

(2) Suitable equivalent geometric imperfections should be used, with values which reflect the possible effects of all type of imperfections, see 5.3.2 and5.3.3, unless these effects are included in the resistance formulae for member design, see section 5.3.4.

(3) The effects of imperfections should be taken into account in the following cases:

a) Global analysis of frames and bracing systems

b) Member design

5.3.2 Imperfections for global analysis

(1) The assumed shape of imperfections may be derived from the elastic buckling mode of a structure in the plane of buckling considered.

(2) Both the possibilities of in and out of plane buckling with symmetric and asymmetric buckling shapes should be taken into account in the most unfavourable direction and form,

(3) The elastic buckling mode should be determined in the following way:

a) The members of the structures should be considered to be loaded by axial forces N_{Ed} only that result from the elastic analysis of the structure for the design loads. Bending moments in the plane of buckling may be neglected in the members.

b) For this force configuration, the critical buckling mode and the force amplifier α_{crit} for elastic critical buckling should be determined.

c) The minimum force amplifier α_u for the above force configuration N_{Ed} to reach the characteristic resistance of the cross-sections of the members N_{uk} without taking buckling into account should be determined.

d) The relative slenderness of the structure is then

$$\bar{\lambda} = \sqrt{\frac{\alpha_u}{\alpha_{crit}}}$$

e) The critical buckling mode shape may be applied as an imperfection with a maximum amplitude of:

$$e_{0d} = \alpha\left(\bar{\lambda} - 0{,}2\right)\frac{W_{el}}{A}\,\frac{1 - \dfrac{\chi\bar{\lambda}^2}{\gamma_{M1}}}{1 - \chi\bar{\lambda}^2} \quad \text{for } \bar{\lambda} > 0{,}2$$

where

α is the imperfection factor for the relevant buckling curve, see Table 6.1 and Table 6.2;

χ is the reduction factor for the relevant buckling curve depending on the cross-section, see 6.3.1;

W_{el} and A are taken as the relevant cross-sectional values at the location where α_u is at its minimum.

(4) For $\bar{\lambda} \leq 0{,}2$ no imperfection need be considered.

(5) In cases where the critical buckling curve has a bow form with the buckling length L the maximum amplitude values in Table 5.1 may be used as a simplification.

Table 5.1: Simplified design values of initial bow imperfection $e_{0,d}$

Buckling curve acc. to Table 6.1	$e_{0,d}$
a_0	L / ???
a	L / 300
b	L / 250
c	L / 200
d	L / 150

Draft note: Values to be checked.

NOTE These values have been derived for $\bar{\lambda}$-values smaller than 2,5.

5.3.3 Imperfection for analysis of bracing systems

(1) The effects of imperfections should be allowed for in the analysis of bracing systems which are required to provide lateral stability within the length of beams or compression members, by means of an equivalent geometric imperfection of the members to be restrained, in the form of an initial bow imperfection:

$$e_0 = \alpha_m \cdot L / 500 \tag{5.2}$$

where L is the span of the bracing system

and $\alpha_m = \sqrt{0{,}5\left(1 + \frac{1}{m}\right)}$

in which m is the number of members to be restrained.

(2) For convenience, the effects of the initial bow imperfections of the members to be restrained by a bracing system, may be replaced by the equivalent stabilising force as shown in Figure 5.1:

$$q = \sum N \cdot 8 \cdot \frac{e_0 + \delta_q}{L^2} \tag{5.3}$$

where δ_q is the inplane deflection of the bracing system due to q plus any external loads

(3) Where the bracing system is required to stabilise a beam of constant height, if not specified in a more accurate way, the force N in Figure 5.1 may be obtained from:

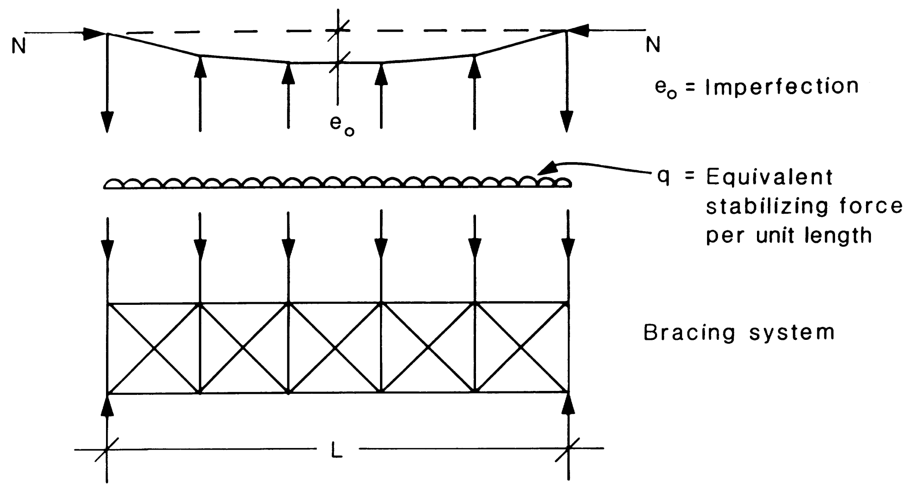
$$N = M / h \qquad (5.4)$$

where M is the maximum moment in the beam

and h is the overall depth of the beam.

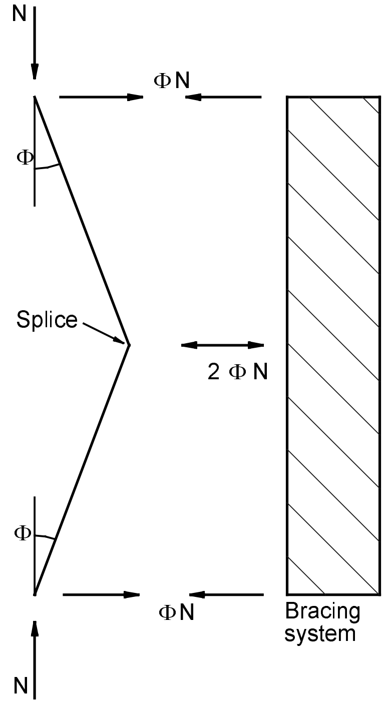
(4) At points where beams or compression members are spliced, it should also be verified that the bracing system is able to resist a local force equal to $\alpha_m \cdot N / 100$ applied to it by each beam or compression member which is spliced at that point, and to transmit this force to the adjacent points at which that beam or compression member is restrained, see Figure 5.2.

(5) When checking for the local force according to clause (4), any external loads acting on bracing systems should also be included, but the forces arising from the imperfection given in (1) may be omitted.



The force N is assumed uniform within the span L of the bracing system. For non-uniform force this is slightly conservative.

Figure 5.1: Equivalent stabilising force



$$\Phi = \alpha_m \Phi_0 \quad : \Phi_0 = 1 / 200$$

$$2 \Phi N = \alpha_m N / 100$$

Figure 5.2: Bracing forces at splices in compression elements

5.3.4 Member imperfections

(1) The imperfections of members are incorporated within the buckling formula given in this Eurocode, see section 6.3.

(2) If the stability of members is accounted for by second order analysis according to 5.2.2(2) method a) for compression members imperfections $e_{0,d}$ according to 5.3.2(3) or 5.3.2(5) should be considered.

(3) For a second order analysis taking account of lateral torsional buckling of a member the imperfections may be adopted as 0,5 $e_{0,d}$, where $e_{0,d}$ is the equivalent initial bow imperfection of the weak axis of the profile considered. In general an additional torsional imperfection need not to be allowed for.

5.4 Calculation of action effects

5.4.1 Methods of analysis considering the effects of deformed geometry

(1)P The effects of the deformed geometry (second-order effects) shall be considered if they increase the action effects significantly, see criteria in 5.2.1.

(2) The effects of shear lag and of local buckling on the stiffness shall be taken into account if this significantly influences the global analysis, see EN 1993-1-5.

(3) The effects of shear lag of the flanges in elastic global analysis may be taken into account according to Part 1-5, 2.3(2) and(3) by the use of an effective width. For simplicity this effective width may be assumed to be uniform over the length of the beam. For each span of a beam the effective width of flanges should be taken as the lesser of the full width and $L_0/8$ per side of the web, where L_0 is the distance between points of zero moments or twice the distance from the support to the end, for a cantilever.

(4) For the global analysis the effect of plate buckling on the stiffness may be taken into account according to Part 1-5, 2.3(4). The effect of plate buckling may be ignored in normal plated structures. If the effective cross-sectional area according to 6.2.2.4 of an element in compression is less than $\boxed{0,5}$ times the gross cross-sectional area, the reduction of the stiffness due to plate buckling should be considered.

(5) The effects of the slip in bolt holes and similar deformations of connection devices like studs and anchor bolts on action effects shall be taken into account, where relevant.

5.4.2 Methods of analysis considering material nonlinearities

5.4.2.1 General

(1) The internal forces and moments may be determined using either

a) elastic global analysis

b) plastic global analysis.

(2) Elastic global analysis may be used in all cases.

(3) Plastic global analysis should be used only where the member cross-sections satisfy the requirements specified in 5.5.3 and where required the joints are able to sustain the plastic resistance for a sufficient rotation capacity, see prEN 1993-1-8.

NOTE The National Annex may define particular conditions for the use of plastic global analysis, e.g. for structures exposed to deep temperatures or where special serviceability or durability criteria apply.

5.4.2.2 Elastic global analysis

(1) Elastic global analysis shall be based on the assumption that the stress-strain behaviour of the material is linear, whatever the stress level is.

(2) For a semi-continuous joint model the rotational stiffness of the joint S_j should be used, see EN 1993-1-8.

NOTE For the choice of a semi-continuous joint model see 5.1.2(5).

(3) Action effects may be calculated according to elastic global analysis even where the resistance of a cross section is based on its plastic resistance, see 5.5.4.

5.4.2.3 Plastic global analysis

(1) Plastic global analysis allows for the effects of material non-linearity in calculating the action effects of a structural system. The behaviour should be modelled

- either by elasto-plastic analysis with plastified sections and/or joints as plastic hinges or
- or by non-linear plastic analysis considering the partial plastification of members in plastic zones.

(2) When plastic global analysis is used, lateral restraint should be provided at all plastic hinge and plastic zone locations at which rotations may occur under any load case.

(3) The lateral restraint should be provided at the compression flange within a distance along the member from the theoretical plastic hinge location not exceeding 1,5 times the width of the flange.

(4) The bi-linear stress-strain relationship indicated in Figure 5.3 may be used for the grades of structural steel specified in section 3. Alternatively, a more precise relationship may be adopted, see EN 1993-1-5.

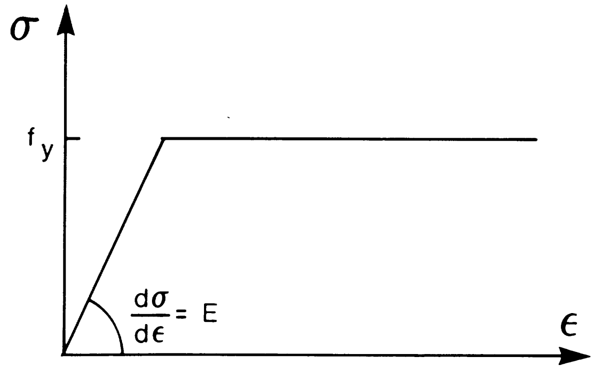


Figure 5.3: Bi-linear stress-strain relationship

(5) System stability should be checked at the intermittent stages of plastification.

(6) For a semi continuous joint the rotation stiffness S_j of the joint should be considered, see EN 1993-1-8.

(7) If no effects of the deformed geometry (second-order effects) have to be considered, see 5.2.1rigid-plastic analysis, i.e. plastic global analysis neglecting the elastic behaviour may be used. In this case joints are classified only by strength, see prEN 1993-1-8.

5.5 Classification of cross sections

5.5.1 Basis

(1) When plastic global analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity to enable the required redistribution of bending moments to develop.

(2) When elastic global analysis is used, any class of cross-section may be used for the members, provided that the design of the members takes into account the possible limits on the resistance of cross-sections due to local buckling, see 6.2.2.4.

5.5.2 Classification

(1) Four classes of cross-sections are defined, as follows:

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required for plastic hinges.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity.
- Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

(2) Effective widths may be used in Class 4 cross-sections to make the necessary allowances for reductions in resistance due to the effects of local buckling, see EN 1993-1-5, 5.2.2.

(3) The classification of cross-section depends on the proportions of each of its compression elements.

(4) Compression elements include every element of a cross-section which is either totally or partially in compression under the load combination considered.

(5) The various compression elements in a cross-section (such as a web or flange) can, in general, be in different classes.

(6) A cross-section is classified by quoting the highest (least favourable) class of its compression elements. Exceptions are specified in 5.5.4(5) and 5.5.4(6).

(7) Alternatively the classification of a cross-section may be defined by quoting both the flange classification and the web classification.

(8) The limiting proportions for Class 1, 2, and 3 compression elements should be obtained from Table 5.2. An element which fails to satisfy the limits for Class 3 should be taken as Class 4.

(9) Except as given in (10) Class 4 sections may be treated as Class 3 sections if the width to thickness ratios are less than the limiting proportions for Class 3 obtained from Table 5.2 increased by $\sqrt{\frac{f_y / \gamma_{M0}}{\sigma_{com,Ed}}}$, where $\sigma_{com,Ed}$ is the maximum design compressive stress in the element taken from the second order analysis where necessary.

(10) However, when verifying the design buckling resistance of a member using section 6.3, the limiting proportions for Class 3 should always be obtained from Table 5.2.

5.5.3 Cross-section requirements for plastic global analysis

(1) At plastic hinge locations, the cross-section of the member which contains the plastic hinge shall have a rotation capacity of not less than the required rotation at that plastic hinge location.

(2) The requirement of a sufficient rotation capacity can be assumed to be satisfied if all members containing plastic hinges have Class 1 cross-sections at the plastic hinge location. For special systems requiring extraordinary high rotation values a direct verification of the rotation capacity comparing the required rotation determined for a system with the available rotation of a section or a joint is necessary.

(3) Where the cross-sections of the members vary along their length, the following additional criteria should be satisfied:

a) Adjacent to plastic hinge locations, the thickness of the web should not be reduced for a distance along the beam from the plastic hinge location of at least 2d, where d is the clear depth of the web at the plastic hinge location. Web stiffeners should be provided within a distance of d/2 of the hinge location when the transverse force at the hinge exceeds 10% of the shear resistance of the cross section.

b) Adjacent to plastic hinge locations, the compression flange should be Class 1 and any fastener holes in tension should satisfy 6.2.5(4) for a distance along the beam from the plastic hinge location of not less than the greater of:

   - 2d, where d is as defined in a)
   - the distance to the point at which the moment in the beam has fallen to 0,8 times the plastic moment resistance at the point concerned.

c) Elsewhere the compression flange should be Class 1 or Class 2.

d) In cases where sophisticated methods of plastic global analysis are used which considers the real stress and strain behaviour along the member, local and global buckling c) has not to be considered.

5.5.4 Cross-section requirements when elastic global analysis is used

(1) When elastic global analysis is used, the role of cross-section classification is to identify the extent to which the resistance of a cross-section is limited by its local buckling resistance.

(2) When all the compression elements of a cross-section are Class 2, the cross-section may be taken as capable of developing its full plastic resistance moment.

(3) When all the compression elements of a cross-section are Class 3, its resistance should be based on an elastic distribution of stresses across the cross-section, limited to the yield strength at the extreme fibres.

(4) When yielding first occurs on the tension side of the neutral axis, the plastic reserves of the tension zone may be utilised when determining the resistance of a Class 3 cross-section.

(5) Cross-sections with a Class 3 web and Class 1 or 2 flanges may be treated as effective Class 2 cross-sections with an effective web in accordance with Figure 5.4. The proportion of the web in compression should be replaced by an element of $20 \cdot \varepsilon \cdot t_w$ adjacent to the compression flange, with another element of $20 \cdot \varepsilon \cdot t_w$ adjacent to the plastic neutral axis of the effective cross-section.

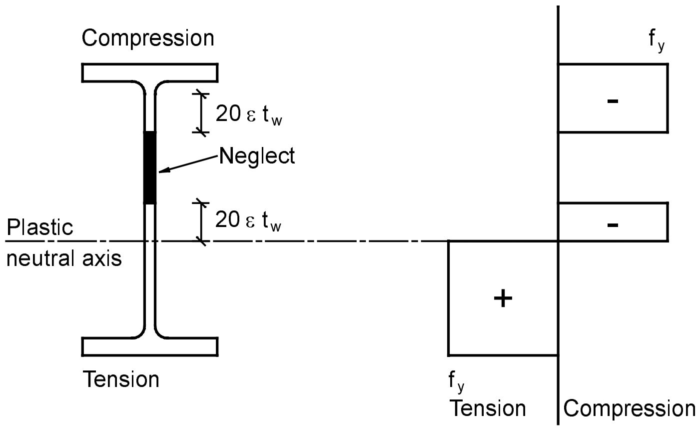
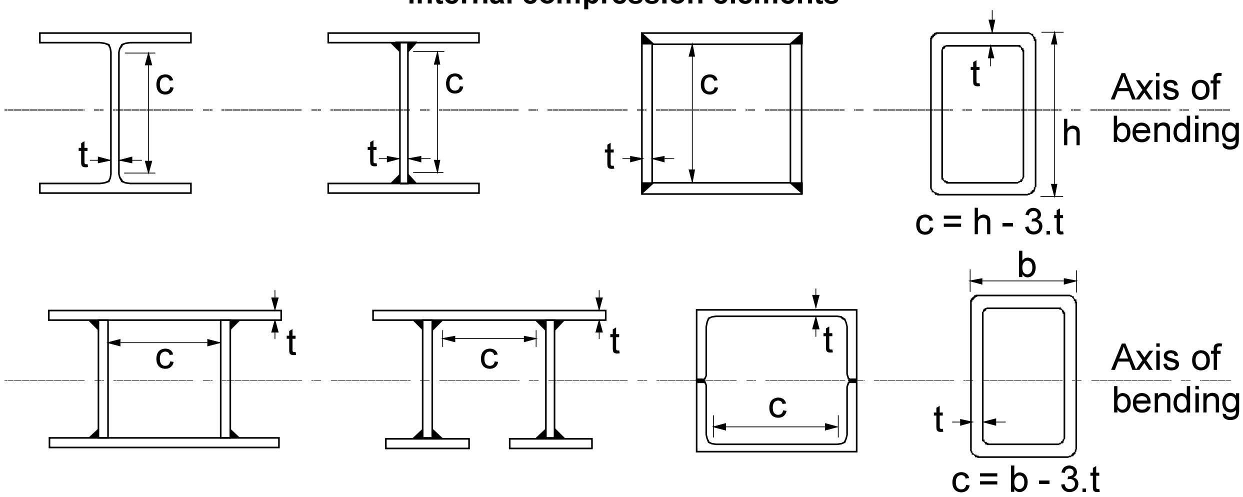


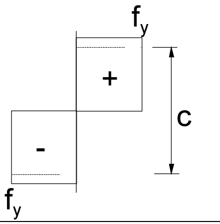
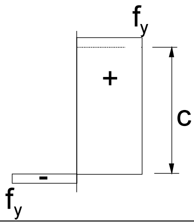
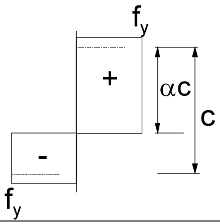
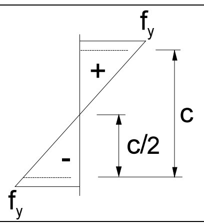
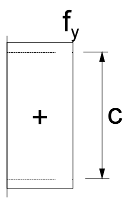
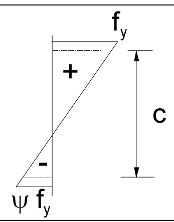
Figure 5.4: Effective class 2 web

(6) When the web is considered to resist shear forces only and does not contribute to the bending and normal force resistance of the cross-section, the cross-section may be designed as Class 2, 3 or 4 sections, depending only on the flange class.

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression elements

Internal compression elements



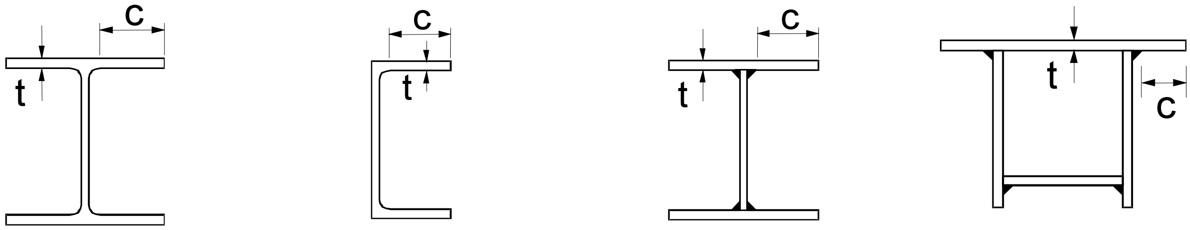
Class	Element subject to bending	Element subject to compression	Element subject to bending and compression
Stress distribution in elements (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0,5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0,5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $c/t \le \dfrac{41,5\varepsilon}{\alpha}$
Stress distribution in elements (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0,67 + 0,33\psi}$ when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

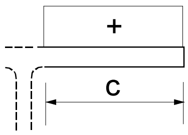
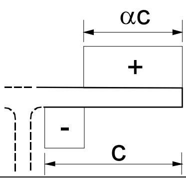
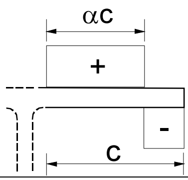
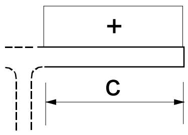
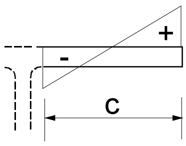
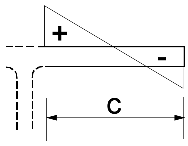
*) This c/t value refers to the utilisation of plastic reserves in the tension zone, see 5.5.4(5), therefore ψ refers to the strain ratio.

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression elements

Outstand flanges



Rolled sections — Welded sections

Class	Element subject to compression	Element subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in elements (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \dfrac{9\varepsilon}{\alpha}$	$c/t \leq \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \dfrac{10\varepsilon}{\alpha}$	$c/t \leq \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in elements (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see Part 1-5	

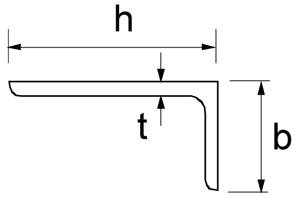
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

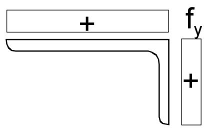
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression elements

<table>
<tr><td colspan="7">

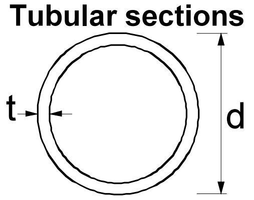
Angles

Refer also to “Outstand flanges” (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components
</td></tr>
<tr><td>Class</td><td colspan="6">Section in compression</td></tr>
<tr><td>Stress distribution across section (compression positive)</td><td colspan="6">
</td></tr>
<tr><td>3</td><td colspan="6">$h/t \leq 15\varepsilon : \frac{b+h}{2t} \leq 11{,}5\varepsilon$</td></tr>
<tr><td colspan="7">

Tubular sections



</td></tr>
<tr><td>Class</td><td colspan="6">Section in bending and/or compression</td></tr>
<tr><td>1</td><td colspan="6">$d/t \leq 50\varepsilon^2$</td></tr>
<tr><td>2</td><td colspan="6">$d/t \leq 70\varepsilon^2$</td></tr>
<tr><td>3</td><td colspan="6">$d/t \leq 90\varepsilon^2$</td></tr>
<tr><td rowspan="3">$\varepsilon = \sqrt{235/f_y}$</td><td>$f_y$</td><td>235</td><td>275</td><td>355</td><td>420</td><td>460</td></tr>
<tr><td>ε</td><td>1,00</td><td>0,92</td><td>0,81</td><td>0,75</td><td>0,71</td></tr>
<tr><td>ε^2</td><td>1,00</td><td>0,85</td><td>0,66</td><td>0,56</td><td>0,51</td></tr>
</table>

6 Ultimate limit states

6.1 General

(1) The partial factors γ_M as defined in 2.4.3 are applied to the various characteristic values of resistance in this section as follows:

- resistance of cross-sections to excessive yielding including local buckling (depending on f_y): γ_{M0}
- resistance of members to member buckling: γ_{M1}
- resistance of cross-sections in tension to fracture (depending on f_u): γ_{M2}
- resistance of joints: see Part 1-8

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

$\gamma_{M0} = 1,00$

$\gamma_{M1} = 1,10$

$\gamma_{M2} = 1,25$

6.2 Resistance of cross-sections

6.2.1 General

(1) This clause covers the resistance of member cross-sections, which are limited by one of the following cases:

- the resistance of the gross cross-section to excessive yielding
- the resistance of the net section at holes for fasteners to fracture in tension

As far as shear lag effects or local buckling effects of class 4 sections are concerned these effects are included by the effective width according to EN 1993-1-5. Shear buckling effects should also be considered according to EN 1993-1-5.

(2) The design value of an action effect in each cross section shall not exceed the corresponding design resistance and if several action effects act simultaneously the combined effect shall not exceed the resistance for that combination.

(3) This requirement (2) shall be fulfilled in all cross-sections corresponding to the classification of the cross-section.

(4) Elastic verification according to the elastic resistance is possible for all cross sectional classes provided the effective cross sectional properties are used for class 4.

(5) If not otherwise specified, for the elastic verification the following yield criterion should be satisfied:

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2+\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2-\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)+3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2\leq 1 \qquad (6.1)$$

where:

$\sigma_{x,Ed}$ is the design value of the local longitudinal stress at the point of consideration

$\sigma_{z,Ed}$ is the design value of the local transverse stress at the point of consideration

τ_{Ed} is the design value of the local shear stress at the point of consideration

(6) The plastic resistance of a cross-sections shall be verified by finding a stress distribution which equilibrates the internal forces and moments without exceeding the yield strength, provided that this stress distribution is feasible, considering the associated plastic deformations.

(7) As a conservative approximation for all cross sectional classes a linear summation of the utilisation ratios for each stress resultant may be used. For example for the combination of N_{Ed} , $M_{y,Ed}$ and $M_{z,Rd}$ by using the following criteria:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1 \qquad (6.2)$$

where N_{Rd}, $M_{y,Rd}$ and $M_{z,Rd}$ are the design values of the resistance depending on the cross sectional classification.

6.2.2 Section properties

6.2.2.1 Gross cross-section

(1) The properties of the gross cross-section shall be determined using the nominal dimensions. Holes for fasteners need not be deducted, but allowance shall be made for larger openings. Splice materials shall not be included.

6.2.2.2 Net area

(1) The net area of a cross-section shall be taken as its gross area less appropriate deductions for all holes and other openings.

(2) When calculating net section properties, the deduction for a single fastener hole should be the gross cross-sectional area of the hole in the plane of its axis. For countersunk holes, appropriate allowance should be made for the countersunk portion.

(3) Provided that the fastener holes are not staggered, the total area to be deducted for fastener holes should be the maximum sum of the sectional areas of the holes in any cross-section perpendicular to the member axis (see failure plane ② in Figure 6.1).

(4) When the fastener holes are staggered, the total area to be deducted for fasteners shall be the greater of:

a) the deduction for non-staggered holes given in (3)

b) $$t \cdot \left(n \cdot d - \sum \frac{s^2}{4 \cdot p} \right) \qquad (6.3)$$

where:

s is the staggered pitch, the spacing of the centres of two consecutive holes in the chain measured parallel to the member axis;

p is the spacing of the centres of the same two holes measured perpendicular to the member axis;

t is the thickness;

n is the number of holes extending in any diagonal or zig-zag line progressively across the member or part of the member, see Figure 6.1.

(5) In an angle or other member with holes in more then one plane, the spacing p shall be measured along the centre of thickness of the material (see Figure 6.2).

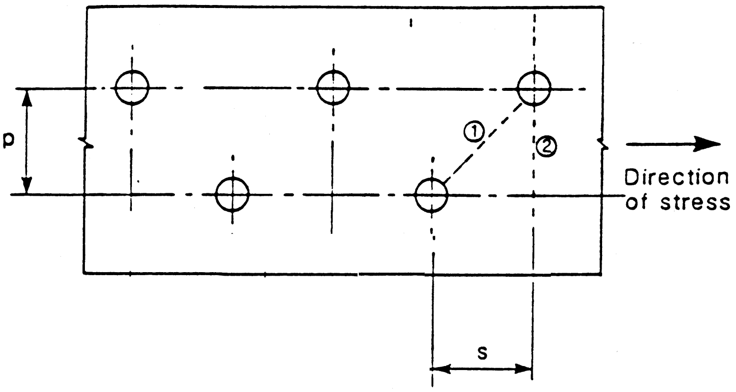


Figure 6.1: Staggered holes

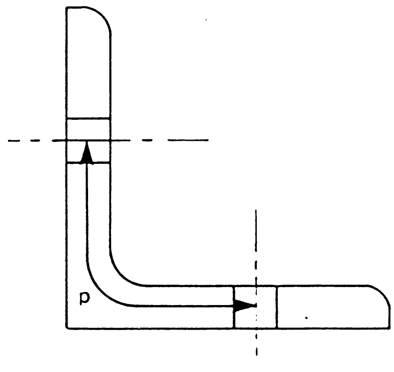


Figure 6.2: Angles with holes in both legs

6.2.2.3 Shear lag effects

(1) The calculation of the effective widths is covered in EN 1993-1-3 for cold formed thin gauge members and in EN 1993-1-5 for planar plated structures

(2) In case of Class 4 sections the interaction between shear lag and local buckling shall be considered according to EN 1993-1-3 for cold formed thin gauge members and EN 1993-1-5 for planar plated structures.

6.2.2.4 Effective cross-section properties of Class 4 cross-sections

(1) The effective cross-section properties of Class 4 cross-sections should be based on the effective widths of the compression elements.

(2) The effective widths of planar compression elements should be obtained from prEN 1993-1-5.

(3) For circular hollow sections with class 4 cross sections see EN 1993-1-6.

(4) When the cross section is subjected to an axial force, the method given in EN 1993-1-5 should be used to determine the possible shift e_N of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross section and the resulting additional moment:

$$\Delta M_{Ed} = N_{Ed} \cdot e_N \tag{6.4}$$

where N_{Ed} is positive for compression.

6.2.3 Tension

(1) The design value of the tension force N_{Ed} at each cross section shall satisfy:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1{,}0 \tag{6.5}$$

(2) For sections with holes the design tension resistance $N_{t,Rd}$ should be taken as the smaller of:

a) the design plastic resistance of the gross cross-section

$$N_{pl,Rd} = A \frac{f_y}{\gamma_{M0}} \tag{6.6}$$

b) the design ultimate resistance of the net cross-section at holes for fasteners

$$N_{u,Rd} = 0{,}9 \cdot A_{net} \frac{f_u}{\gamma_{M2}} \tag{6.7}$$

(3) When capacity design is requested, see EN 1998, the design plastic resistance $N_{pl,Rd}$ (as given in 6.2.3 a)) should be less than the design ultimate resistance of the net section at fasteners holes $N_{u,Rd}$ (as given in 6.2.3 b)).

(4) In category C connections (see prEN 1993-1-8, 3.1.1(4)), the design plastic resistance of the net section at holes for fasteners $N_{net,Rd}$ should not be taken as more than:

$$N_{net,Rd} = A_{net} \frac{f_y}{\gamma_{M0}} \tag{6.8}$$

(5) For angles connected through one leg, see also Part 1-8, 3.6.3. Similar consideration should also be given to other type of sections connected through outstands such as T-sections and channels.

6.2.4 Compression

(1) The design value of the compression force N_{Ed} at each cross-section shall satisfy:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1{,}0 \tag{6.9}$$

(2) The design resistance of the cross-section for uniform compression $N_{c,Rd}$ shall be determined as follows:

Class 1, 2 or 3 cross-sections: $N_{c,Rd} = A\, f_y / \gamma_{M0}$

Class 4 cross-sections: $N_{c,Rd} = A_{eff}\, f_y / \gamma_{M0}$

(3) Fastener holes need not be allowed for in compression members, as far as they are filled by fasteners, except for oversize and slotted holes.

(4) In the case of unsymmetrical Class 4 sections, the method given in 6.2.9.3 should be used to allow for the additional moment ΔM due to the eccentricity of the centroidal axis of the effective section, see 6.2.2.4(4).

6.2.5 Bending moment

(1) The design value of the bending moment M_{Ed} at each cross-section shall satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1{,}0 \tag{6.10}$$

(2) The design resistance for bending about one principal axis of a cross-section without holes for fasteners is determined as follows:

Class 1 or 2 cross sections: $M_{c,Rd} = W_{pl}\ f_y / \gamma_{M0}$

Class 3 cross-sections: $M_{c,Rd} = W_{el,min}\ f_y / \gamma_{M0}$

Class 4 cross-sections: $M_{c,Rd} = W_{eff,min}\ f_y / \gamma_{M0}$

where $W_{el,min}$ and $W_{eff,min}$ corresponds to the fibre with the maximum elastic stress.

(3) For bending about both axes, the methods given in 6.2.9 should be used.

(4) Fastener holes in the tension flange may be ignored provided that for the tension flange:

$$0.9\left[\frac{A_{f,net}}{A_f}\right] \geq \left[\frac{f_y}{f_u}\right]\left[\frac{\gamma_{M2}}{\gamma_{M0}}\right] \tag{6.11}$$

where A_f is the area of the tension flange.

NOTE The criterion in (4) provides capacity design in the region of plastic hinges.

(5) Fastener holes in tension zone of the web need not be allowed for, provided that the limit given in (4) is satisfied for the complete tension zone comprising the tension flange plus the tension zone of the web.

(6) Fastener holes in compression zone of the cross-section need not be allowed for, as far as they are filled by fasteners, except for oversize and slotted holes.

6.2.6 Shear

(1) The design value of the shear force V_{Ed} at each cross-section shall satisfy:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1,0 \tag{6.12}$$

where $V_{c,Rd}$ is the design shear resistance. For plastic design $V_{c,Rd}$ is the design plastic shear resistance $V_{pl,Rd}$ as given in (2). For elastic design $V_{c,Rd}$ is the design elastic shear resistance as given in (5).

(2) The design plastic shear resistance is given by:

$$V_{pl,Rd} = A_v \left(f_y / \sqrt{3}\right) / \gamma_{M0} \tag{6.13}$$

where A_v is the shear area.

(3) The shear area A_v may be taken as follows:

a) rolled I and H sections, load parallel to web $A - 2bt_f + \left(t_w + 2r\right) t_f$

b) rolled channel sections, load parallel to web $A - 2bt_f + \left(t_w + r\right) t_f$

c) welded I, H and box sections, load parallel to web $\sum\left(h_w t_w\right)$

d) welded I, H, channel and box sections, load parallel to flanges $A - \sum\left(h_w t_w\right)$

e) rolled rectangular hollow sections of uniform thickness:

load parallel to depth $Ah/(b+h)$

load parallel to width $Ab/(b+h)$

f) circular hollow sections and tubes of uniform thickness $2A/\pi$

where

A is the cross-section area;

b is the overall breadth;

h is the overall depth;

h_w is the depth of the web;

r is the root radius;

t_f is the flange thickness;

t_w is the web thickness (If the web thickness in not constant, t_w should be taken as the minimum thickness.).

(4) As an alternative to (3) c) the shear area A_v for welded I, H or box sections, load parallel to web may be taken as follows:

$A_v = 1{,}20\ h_w\, t_w$ for S235, S275 and S355

$A_v = 1{,}05\ h_w\, t_w$ for S420 and S460

provided that end posts are used and the welds between web and flanges are designed for the corresponding shear flow.

Draft note: ... to be checked.

(5) The design elastic shear resistance $V_{c,Rd}$ should be such that all points in the cross-section satisfy the following criterion:

$$\frac{\tau_{Ed}}{f_y / \left(\sqrt{3} \cdot \gamma_{M0}\right)} \leq 1{,}0 \qquad (6.14)$$

where τ_{Ed} shall be obtained from: $\tau_{Ed} = \dfrac{V_{Ed} \cdot S}{I \cdot t}$ (6.15)

where V_{Ed} is the design value of the shear force

S is first moment of area

I is second moment of area

t is the thickness at the examined point

(6) For I- or H-Profiles where the line of application of the transverse force complies with the centreline of the web the shear stress in the web may be taken as:

$$\tau_{md} = \frac{V_{Ed}}{A_w} \quad \text{if } A_{fl} / A_w \geq 0{,}6 \qquad (6.16)$$

where A_{fl} is the area of one flange;

A_w is the area of the web: $A_w = h_w \cdot t_w$.

Draft note: The following clauses have to be updated according to prEN 1993-1-5.

(7) In addition the shear buckling resistance shall be verified as specified according to EN 1993-1-5.

$$\frac{h_w}{t_w} > 31 \frac{\varepsilon}{\eta} \sqrt{k_\tau} \qquad (6.17)$$

For webs without intermediate stiffeners (6.17) can be specified as follows:

$$\frac{h_w}{t_w} > 72 \frac{\varepsilon}{\eta} \qquad (6.18)$$

where k_τ is the buckling factor for shear, see EN 1993-1-5

$\eta = 1{,}20 \dfrac{\gamma_{M1}}{\gamma_{M0}}$ for S 235, S 275 and S 355

$\eta = 1{,}05 \dfrac{\gamma_{M1}}{\gamma_{M0}}$ for S 420 and S 460

$\varepsilon = \sqrt{\dfrac{235}{f_y}}$

(8) Fastener holes need not to be allowed for in the shear verification except in verifying the design shear resistance at connection zones as given in EN 1993-1-8.

6.2.7 Torsion

(1) The design value of the torsional moment T_{Ed} at each cross-section should satisfy:

$$\frac{T_{Ed}}{T_{Rd}} \leq 1{,}0 \qquad (6.19)$$

(2) The total internal torque T_{Ed} at any cross- section should generally be divided into two parts such that:

$$T_{Ed} = T_{v,Ed} + T_{w,Ed} \qquad (6.20)$$

where

$T_{v,Ed}$ is the St. Venant torsion;

$T_{w, Ed}$ is the warping torsion.

Draft note:	For the St. Venant torsion the index v should be replaced in order to avoid mix-ups with the index for shear .

(3) The values of $T_{v,Ed}$ and $T_{w,Ed}$ at any cross-section may be determined from T_{Ed} by elastic analysis, taking account of the section properties of the member, the conditions of restraint at the supports and the distribution of the actions along the member.

(4) As a simplification, in the case of a member with a closed hollow cross-section, such as a structural hollow section, it may be assumed that the effects of warping can be neglected.

(5) The following stresses due to torsion should be taken into account:

- the shear stresses $\tau_{v,Ed}$ due to St. Venant torsion
- the direct stresses $\sigma_{w,Ed}$ due to the bimoment B_{Ed} and shear stresses $\tau_{w,Ed}$ due to warping torsion $T_{w,Ed}$

(6) For plastic resistance only torsion necessary for equilibrium should be considered.

(7) For the calculation of the resistance T_{Rd} a design shear stress of $f_y / (\gamma_{M0} \cdot \sqrt{3})$ should be assumed.

(8) For combined shear forces and torsional moments in Class 1 or 2 sections the plastic shear stress should be limited by verifying the resistance of the cross-section $V_{pl,TEd,Rd}$ to combined shear force and torsional moment using:

$$\frac{V_{Ed}}{V_{pl,TEd,Rd}} \leq 1 \tag{6.21}$$

in which $V_{pl,TEd,Rd}$ is given by the following:

– for an I or H section:

$$V_{pl,TEd,Rd} = \sqrt{1 - \frac{\tau_{v,Ed}}{1{,}25\left(f_y/\sqrt{3}\right)/\gamma_{M0}}}\ V_{pl,Rd} \tag{6.22}$$

– for a channel section:

$$V_{pl,TEd,Rd} = \left[\sqrt{1 - \frac{\tau_{v,Ed}}{1{,}25\left(f_y/\sqrt{3}\right)/\gamma_{M0}}} - \frac{\tau_{w,Ed}}{\left(f_y/\sqrt{3}\right)/\gamma_{M0}}\right] V_{pl,Rd} \tag{6.23}$$

– for a structural hollow section:

$$V_{pl,TEd,Rd} = \left[1 - \frac{\tau_{v,Ed}}{\left(f_y/\sqrt{3}\right)/\gamma_{M0}}\right] V_{pl,Rd} \tag{6.24}$$

6.2.8 Bending and shear

(1) When the shear force is present allowance shall be made for its effect on the moment resistance.

(2) When the shear force is less than half the plastic shear resistance the effect on the moment resistance may be neglected.

NOTE For small values of shear force the reduction of the plastic resistance moment is so small that it is counter-balanced by strain hardening and may be neglected.

(3) Otherwise the reduced resistance should be taken as the design resistance of the cross-section, calculated using a reduced strength $(1-\rho)f_y$ for the shear area, but not more than $M_{c,Rd}$,

where $\rho = \left(\frac{2\,V_{Ed}}{V_{pl,Rd}} - 1\right)^2$ and $V_{pl,Rd}$ is obtained from 6.2.6(2).

(4) In the presence of torsion ρ should be obtained from $\rho = \left(\frac{2\,V_{Ed}}{V_{pl,TEd,Rd}} - 1\right)^2$, see 6.2.7

(5) The reduced design plastic resistance moment allowing for the shear force may alternatively be obtained for cross-sections with equal flanges and bending about the major axis as follows:

$$M_{V,Rd} = \left[W_{pl} - \frac{\rho A_v^2}{4\,t_w}\right] \cdot \frac{f_y}{\gamma_{M0}} \quad \text{but} \quad M_{V,Rd} \leq M_{c,Rd} \tag{6.25}$$

where $M_{c,Rd}$ is obtained from 6.2.5(2).

6.2.9 Bending and axial force

6.2.9.1 Class 1 and 2 cross-sections

(1) When an axial force is present, allowance shall be made for its effect on the plastic moment resistance.

(2) For class 1 and 2 cross sections, the following criterion should be satisfied:

$$M_{Ed} \leq M_{N,Rd} \tag{6.26}$$

For a plate without bolt holes, the reduced design plastic resistance moment $M_{N,Rd}$ is given by:

$$M_{N,Rd} = M_{pl,Rd}\left[1-\left(N_{Ed} / N_{pl,Rd}\right)^2\right] \tag{6.27}$$

(3) For bending about the y-y-axis, allowance should be made for the effect of the axial force on the plastic resistance moment when the following criteria are satisfied:

$$N_{Ed} > 0{,}25\, N_{pl,Rd}\text{, or} \tag{6.28}$$

$$N_{Ed} > 0{,}5\, h_w\, t_w\, f_{yd}\text{, whichever is smaller} \tag{6.29}$$

Similarly, for bending about the z-z-axis, allowance should be made for the effect of the axial force when:

$$N_{Ed} > h_w\, t_w\, f_{yd} \tag{6.30}$$

(4) For cross-sections without bolt holes, the following approximations may be used for standard rolled I or H sections and for welded I or H sections with equal flanges:

$$M_{Ny.Rd} = M_{pl.y.Rd}\,(1-n)/(1-0{,}5a) \text{ but } M_{Ny.Rd} \leq M_{ply.Rd} \tag{6.31}$$

$$\text{for } n \leq a: \quad M_{Nz.Rd} \leq M_{pl.z.Rd} \tag{6.32}$$

$$\text{for } n > a: \quad M_{Nz.Rd} = M_{pl.z.Rd}\left[1-\left[\frac{n-a}{1-a}\right]^2\right] \tag{6.33}$$

where $n = N_{Ed} / N_{pl.Rd}$

$a = (A-2bt_f)/A$ but $a \leq 0{,}5$

For cross-sections without bolt holes, the following approximations may be used for rectangular structural hollow sections of uniform thickness and for welded box sections with equal flanges and equal webs:

$$M_{Ny.Rd} = M_{pl.y.Rd}\,(1 - n)/(1 - 0{,}5a_w) \text{ but } M_{Ny.Rd} \leq M_{pl.y.Rd} \tag{6.34}$$

$$M_{Nz.Rd} = M_{pl.z.Rd}\,(1 - n)/(1 - 0{,}5a_f) \text{ but } M_{Nz.Rd} \leq M_{pl.z.Rd} \tag{6.35}$$

where $a_w = (A - 2bt)/A$ but $a_w \leq 0{,}5$ for hollow sections

$a_w = (A-2bt_f)/A$ but $a_w \leq 0{,}5$ for welded box sections

$a_f = (A - 2ht)/A$ but $a_f \leq 0{,}5$ for hollow sections

$a_f = (A-2ht_w)/A$ but $a_f \leq 0{,}5$ for welded box sections

(5) For bi-axial bending the following approximate criterion may be used:

$$\left[\frac{M_{y,Ed}}{M_{Ny,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{Nz,Rd}}\right]^{\beta} \leq 1 \tag{6.36}$$

in which α and β are constants, which may conservatively be taken as unity, otherwise as follows:

- I and H sections:

  $\alpha = 2\ ;\ \beta = 5n \quad \text{but}\ \beta \geq 1$

- circular hollow sections:

  $\alpha = 2\ ;\ \beta = 2$

- rectangular hollow sections:

  $\alpha = \beta = \dfrac{1.66}{1-1.13n^2} \qquad \text{but}\ \alpha = \beta \leq 6$

  where $n = N_{Ed} / N_{pl,Rd}$.

6.2.9.2 Class 3 cross-sections

(1) In the absence of shear force, for Class 3 cross-sections the maximum longitudinal stress shall satisfy the criterion:

$$\sigma_{x,Ed} \leq f_y / \gamma_{M0} \tag{6.37}$$

where $\sigma_{x,Ed}$ is the design value of the local longitudinal stress due to moment and axial force

6.2.9.3 Class 4 cross-sections

(1) In the absence of shear force, for Class 4 cross-sections the maximum longitudinal stress $\sigma_{x,Ed}$ calculated using the effective widths of the compression elements (see 5.5.2(2)) shall satisfy the criterion:

$$\sigma_{x,Ed} \leq f_y / \gamma_{M0} \tag{6.38}$$

where $\sigma_{x,Ed}$ is the design value of the local longitudinal stress due to moment and axial force

(2) For cross-sections without holes for fasteners, the above criterion becomes:

$$\frac{N_{Ed}}{A_{eff}\, f_y / \gamma_{M0}} + \frac{M_{y,Ed} + N_{Ed}\, e_{Ny}}{W_{eff,y,min}\, f_y / \gamma_{M0}} + \frac{M_{z,Ed} + N_{Ed}\, e_{Nz}}{W_{eff,z,min}\, f_y / \gamma_{M0}} \leq 1 \tag{6.39}$$

where:

A_{eff} is the effective area of the cross-section when subjected to uniform compression

$W_{eff,min}$ is the effective section modulus of the cross-section when subjected only to moment about the relevant axis corresponding to the fibre with the maximum elastic stress

e_N is the shift of the relevant centroidal axis when the cross-section is subjected to uniform compression, see 6.2.2.4(4)

6.2.10 Bending, shear and axial force

(1) When shear and axial force are present, allowance shall be made for the effect of both shear force and axial force on the resistance moment.

(2) Provided that the design value of the shear force V_{Ed} does not exceed 50% of the design plastic shear resistance $V_{pl.Rd}$ no reduction need be made in combinations of moment and axial force that meet the criteria in 6.2.9.

(3) When V_{Ed} exceeds 50% of $V_{pl.Rd}$ the design resistance of the cross-section to combinations of moment and axial force should be calculated using a reduced yield strength

$(1-\rho)f_y$ for the shear area

where $\rho = (2V_{Ed} / V_{pl.Rd} - 1)^2$ and $V_{pl,Rd}$ is obtained from 6.2.6(2).

6.3 Buckling resistance of members

6.3.1 Compression members

6.3.1.1 Buckling resistance

(1) A compression member shall be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1{,}0 \qquad (6.40)$$

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

$N_{b,Rd}$ is the design buckling resistance of the compression member.

NOTE 1 For hot rolled and welded steel members with the types of cross-section according to Table 6.2 the relevant buckling mode is generally flexural buckling. In case of torsional or flexural-torsional buckling, see 6.3.1.4.

NOTE 2 If the buckling resistance considers non-uniform sections along the member or non-uniform distribution of the compression force, the verification may be applied accordingly for the section considered.

(2) For members with non-symmetric Class 4 sections allowance should be made for the additional moment ΔM due to the eccentricity of the centroidal axis of the effective section, see also 6.2.2.4(4).

(3) The design buckling resistance of a compression member should be taken as:

$$N_{b,Rd} = \chi A \frac{f_y}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross-sections}$$

$$N_{b,Rd} = \chi A_{eff} \frac{f_y}{\gamma_{M1}} \quad \text{for Class 4 cross-sections} \qquad (6.41)$$

where χ is the reduction factor for the relevant buckling mode.

NOTE N_{Rd} is the minimum of $N_{pl,Rd}$ and $N_{b,Rd}$.

6.3.1.2 Buckling curves

(1) For axial compression in members, the value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$, should be determined from:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1{,}0 \tag{6.42}$$

where $\phi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}}$ for Class 1, 2 and 3 cross-sections

$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}}$ for Class 4 cross-sections

α is an imperfection factor

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross sectional properties.

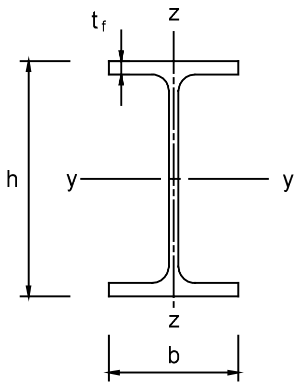
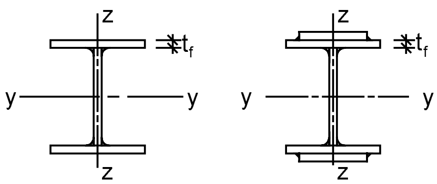
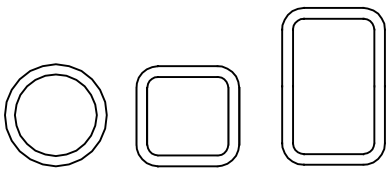
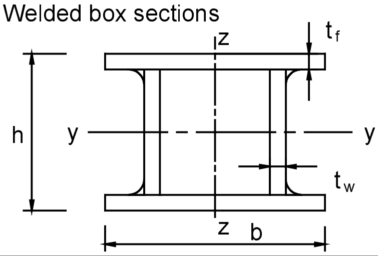
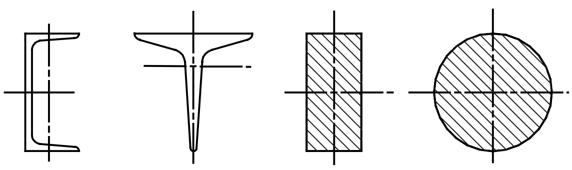
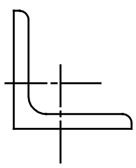
(2) The imperfection factor α corresponding to the appropriate buckling curve should be obtained from Table 6.1 and Table 6.2.

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

(3) Values of the reduction factor χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ may be obtained from Figure 6.3.

Table 6.2: Selection of buckling curve for a cross-section

Cross-section	Limits	Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Rolled I-sections 	$h/b > 1{,}2$ $t_f \le 40mm$	y - y z - z	a b	a_0 a_0
	$40mm < t_f \le 100mm$	y - y z - z	b c	a a
	$h/b \le 1{,}2$ $t_f \le 100mm$	y - y z - z	b c	a a
	$t_f > 100mm$	y - y z - z	d d	c c
Welded I-sections 	$t_f \le 40mm$	y - y z - z	b c	b c
	$t_f > 40mm$	y - y z - z	c d	c d
Hollow sections 	hot rolled	any	a	a_0
	cold formed	any	c	c
Welded box sections 	generally (exept as below)	any	b	b
	thick welds: $a > 0{,}5\ t_f$ $b / t_f < 30$ $h / t_w < 30$	any	c	c
U-,T-and solid sections 		any	c	c
L-sections 		any	b	b

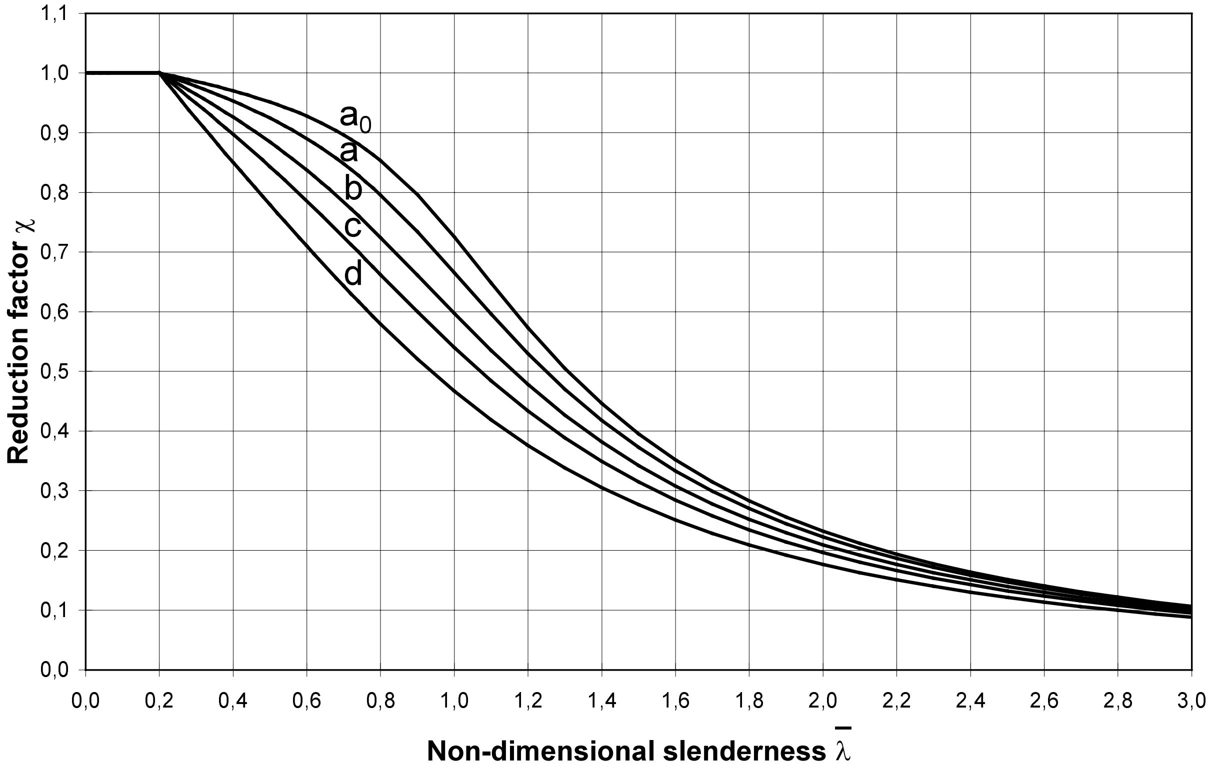


Figure 6.3: Buckling curves

6.3.1.3 Flexural buckling

(1) The non-dimensional slenderness $\bar{\lambda}$ is given by:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1} \qquad \text{for Class 1, 2 and 3 cross-sections}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff}f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{\sqrt{\beta_A}}{\lambda_1} \qquad \text{for Class 4 cross-sections} \tag{6.43}$$

where L_{cr} is the buckling length

i is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section

$$\beta_A = \frac{A_{eff}}{A}$$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93{,}9\varepsilon$$

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad (f_y \text{ in N/mm}^2)$$

(2) The buckling length L_{cr} of a compression member with both ends effectively held in position laterally, may be taken as equal to its system length L.

(3) For flexural buckling the appropriate buckling curve should be determined from Table 6.2.

6.3.1.4 Torsional and torsional-flexural buckling

(1) For members with open cross-sections account shall be taken of the possibility that the resistance of a member to either torsional or torsional-flexural buckling might be less than its resistance to flexural buckling.

(2) The non-dimensional slenderness $\bar{\lambda}_T$ for torsional or torsional-flexural buckling is given by:

$$\bar{\lambda}_T = \sqrt{\frac{Af_y}{N_{cr}}} \quad \text{for Class 1, 2 and 3 cross-sections}$$

$$\bar{\lambda}_T = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \quad \text{for Class 4 cross-sections} \tag{6.44}$$

where $N_{cr} = N_{cr,TF}$ but $N_{cr} < N_{cr,T}$

$N_{cr,TF}$ is the elastic torsional-flexural buckling force

$N_{cr,T}$ is the elastic torsional buckling force

(3) For torsional or torsional-flexural buckling the appropriate buckling curve related to the z-axis may be determined from Table 6.2.

6.3.2 Lateral-torsional buckling of beams

6.3.2.1 Buckling resistance

(1) A laterally unrestrained beam subject to major axis bending shall be verified against lateral-torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0 \tag{6.45}$$

where M_{Ed} is the design value of the moment

$M_{b,Rd}$ is the design buckling resistance moment.

(2) Beams with sufficient restraint to the compression flange are not susceptible to lateral-torsional buckling. In addition, beams with certain types of cross-sections, such as square or circular hollow sections, fabricated circular tubes or square box sections are not susceptible to lateral-torsional buckling.

(3) The design buckling resistance moment of a laterally unrestrained beam should be taken as:

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \tag{6.46}$$

where W_y is the appropriate section modulus of the compression flange as follows

- $W_y = W_{pl,y}$ for Class 1 or 2 cross-sections
- $W_y = W_{el,y}$ for Class 3 cross-sections
- $W_y = W_{eff,y}$ for Class 4 cross-sections

χ_{LT} is the reduction factor for lateral-torsional buckling.

6.3.2.2 Lateral torsional buckling curves

(1) For bending member of constant cross-section, the value of χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$, shall be determined from:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but } \chi_{LT} \leq 1{,}0 \tag{6.47}$$

where $\phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}2\right) + \bar{\lambda}_{LT}^2\right]$

α_{LT} is an imperfection factor

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

M_{cr} is the elastic critical moment for lateral-torsional buckling

Draft note: An alternative proposal is given below which however cannot be justified by tests:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} \quad \text{but} \begin{cases} \chi_{LT} \leq 1{,}0 \\ \chi_{LT} \leq \dfrac{1}{\bar{\lambda}_{LT}^2} \end{cases}$$

$$\phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}4\right) + \beta\bar{\lambda}_{LT}^2\right]$$

$\beta = 0{,}75$

The following requirements are not fulfilled:

- a plateau length of 0,4 is not acceptable for the application for bridges
- the proposal is not in line with the general analytical expression of the European buckling curves that are based on the Ayrton-Perry equation that writes as follows:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \beta\bar{\lambda}^2}}$$

$$\phi = 0{,}5\left[\beta + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \beta\bar{\lambda}^2\right]$$

(2) M_{cr} is based on gross cross sectional properties and takes into account the loading conditions, the real moment distribution, the lateral restraints and for non uniform sections the variation of the cross section.

(3) The imperfection factor α_{LT} corresponding to the appropriate buckling curve shall be obtained from Table 6.3.

Table 6.3: Imperfection factors for lateral torsional buckling curves

Buckling curve	b	c	d
Imperfection factor α	0,34	0,49	0,76

(4) The appropriate buckling curve should be determined from Table 6.4.

Table 6.4: Selection of lateral torsional buckling curve for a cross-section

Cross-section	Limits	Buckling curve
Rolled I-sections	$h/b \leq 2$ $h/b > 2$	**b** **c**
Welded I-sections	$h/b \leq 2$ $h/b > 2$	**c** **d**
Other cross-sections	-	**d**

(4) Values of the reduction factor χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be obtained from Figure 6.3: Buckling curves.

Draft note: A Figure 6.x “Lateral-torsional buckling curves” will be added if the alternative proposal (with modifications) is accepted.

6.3.2.3 Simplified method

(1) Members with I-sections subject to lateral-torsional buckling may be verified by the compression flange as an equivalent compression member.

6.3.3 Lateral torsional buckling of frames

6.3.3.1 General method

(1) For frames composed of beams or columns or beam-columns subject to monoaxial bending and compression, the assessment for lateral torsional buckling out of the plane of the frame may be performed in the following way:

a) For the distribution of action effects on the frame (N_{Ed}, M_{Ed}, V_{Ed}) resulting from the analysis of the frame for the design loads the multiplier α_{crit} of these design loads to reach the elastic critical resistance of the frame with regard to lateral deformations should be determined.

NOTE In this determination the distorsional deformations of the cross section and the effects of any restraint (e.g. by stiffeners or single or continuous connections to flanges) may be considered.

b) For the same distribution of action effects the minimum multiplier α_u of the design loads to reach the characteristic resistance of the frame without taking lateral torsional buckling into account should be determined.

NOTE Both the determination of α_{crit} and α_u may be performed by FEM.

c) The relative slenderness representative for the frame is

$$\bar{\lambda} = \sqrt{\frac{\alpha_u}{\alpha_{crit}}} \tag{6.48}$$

This slenderness should be used to determine the reduction factor χ_{LT} for the relevant lateral torsional buckling curve, see 6.3.2.2.

d) The partial factor applicable to the frame resistance is

$$\gamma_{Mu} = \chi_{LT}\,\alpha_u \tag{6.49}$$

e) The partial factor γ_{Mu} should satisfy

$$\gamma_{Mu} \geq \gamma_{M1} \tag{6.50}$$

NOTE This method may also be used for members instead of the methods given in 6.3.1 and 6.3.2 when the support conditions of the members are irregular or the member is not uniform.

6.3.4 Bending and axial compression

(1) For cases not covered by 6.3.3 and unless second order analysis is carried out using the imperfections as given in 5.3.2, the stability of members with double symmetric cross sections should be checked as given in the following sections, where a distinction is made for:

- members that are not susceptible to torsional deformations, e.g. hollow sections or sections restraint from torsion
- members that are susceptible to torsional deformations, e.g. open cross-sections.

(2) In addition, the resistance of the cross-sections at each end of the member should satisfy the requirements according to 6.2.

(3) Members not susceptible to torsional deformations, which are loaded by combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\dfrac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{C_{my} M_{y,Ed} + \Delta M_{y,Ed}}{\dfrac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{C_{mz} M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

(6.51)

$$\frac{N_{Ed}}{\dfrac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{C_{my} M_{y,Ed} + \Delta M_{y,Ed}}{\dfrac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{C_{mz} M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

where N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the compression force and the moments about the y-y and z-z axis, respectively

ΔM_y, ΔM_z are the moments due to the shift of the centroidal axis according to 6.2.9.3, see Table 6.5,

χ_y and χ_z are the reduction factors due to flexural buckling from 6.3.1

k_{yy}, k_{yz}, k_{zy}, k_{zz} are the interaction factors

$C_{m,y}$ and $C_{m,z}$ are the equivalent uniform moment factors.

Table 6.5: Values for $N_{R,k} = f_y A$, $M_{R,ki} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	3	4
A	A	A_{eff}
W_y	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{el,z}$	$W_{eff,z}$
ΔM_y	0	$e_{N,y} N_{Ed}$
ΔM_z	0	$e_{N,z} N_{Ed}$

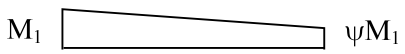
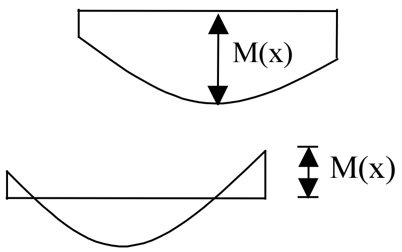
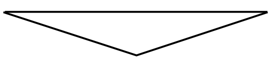
(4) The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} and the equivalent uniform moment factors C_{my} and C_{mz} may be obtained from Table 6.6 and Table 6.7.

NOTE The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} and the equivalent uniform moment factors C_{my} and C_{mz} have been derived for elastic design (i.e. for class 3 and class 4 section). They may also be applied to class 1 and class 2 sections when for these sections elastic resistances are used. Interaction factors for using plastic resistances of cross sections may be given in the National Annex. Recommendations for such factors are given in the informative annex A.

Table 6.6: Amplification factors

k_{yy}	$\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$
k_{yz}	$\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$
k_{zy}	$\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$
k_{zz}	$\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$
Hereby $\mu_y = \dfrac{1-\dfrac{N_{Ed}}{N_{cr,y}}}{1-\chi_y \dfrac{N_{Ed}}{N_{cr,y}}}$ $\quad \mu_z = \dfrac{1-\dfrac{N_{Ed}}{N_{cr,z}}}{1-\chi_z \dfrac{N_{Ed}}{N_{cr,z}}}$	

Table 6.7: Equivalent uniform moment factors

Moment diagram	$C_{m,i}$
 $-1 \leq \psi \leq 1$	$C_{m,i} = 0{,}79 + 0{,}21\psi_i + 0{,}36(\psi_i - 0{,}33)\dfrac{N_{Ed}}{N_{cr.i}}$
	$C_{m,i} = 1 + \left(\dfrac{\pi^2 EI\|_i \delta_x\|}{L^2\|M_x\|} - 1\right)\dfrac{N_{Ed}}{N_{cr.i}}$ M(x) is the maximum first order moment δ_x is the maximum first order member displacement
 	$C_{m,i} = 1 - 0{,}18\dfrac{N_{Ed}}{N_{cr.i}}$ $C_{m,i} = 1 + 0{,}03\dfrac{N_{Ed}}{N_{cr.i}}$

(5) Members susceptible to torsional deformations, which are loaded by combined bending and axial compression should satisfy

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{C_{m,LT}M_{y,Ed} + \Delta M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{C_{mz}M_{z,Ed} + \Delta M_{y,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

(6.52)

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{C_{m,LT}M_{y,Ed} + \Delta M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{C_{mz}M_{z,Ed} + \Delta M_{y,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

where symbols are as in (3) and

χ_{LT} is the reduction factor due to lateral torsional buckling from 6.3.2

$C_{m,LT}$ is the equivalent moment factor for LT-buckling.

(6) The equivalent moment factor $C_{m,LT}$ should be obtained from

$$C_{m,LT} = k_{LT}\left[C_{m,y} + (1 - C_{m,y})\frac{\sqrt{\varepsilon_y}\,a_{LT}}{1 + \sqrt{\varepsilon_y}\,a_{LT}}\right] \quad (6.53)$$

where $C_{m,y}$ is the relevant factor obtained from Table 6.7

with I_y = second moment of area about the y-y axis

I_t = St. Venant torsion constant

$$k_{LT} = C_{m,y}^2 \frac{a_{LT}}{\sqrt{\left(1 - \frac{N_{Ed}}{N_{crit,z}}\right)\left(1 - \frac{N_{Ed}}{N_{crit,T}}\right)}}$$

with $N_{crit,z}$ = elastic flexural buckling force about the z-z axis

$N_{crit,T}$ = elastic torsional buckling force

$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A}{W_{ely}}$ for class 1, 2 and 3 cross-sections

$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A_{eff}}{W_{eff,y}}$ for class 4 cross-sections

NOTE The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} and the equivalent uniform moment factors $C_{m,LT}$ and $C_{m,z}$ have been derived for elastic design (i.e. for class 3 and class 4 sections). They may also be applied to class 1 and class 2 section when for these sections elastic resistances are used. Interaction factors for using plastic resistances of cross sections may be given in the National Annex. Recommendations for such factors are given in the informative annex A.

6.4 Built-up compression members

6.4.1 General

(1) Built-up compression members that are uniform may be designed with the following model, see Figure 6.4.

1. The member may be considered as a column with a bow imperfection $e_0 = \frac{\ell}{500}$
2. The elastic deformations of lacings or battenings, see Figure 6.4, may be considered by a continuous (smeared) shear stiffness S_V of the column.

(2) The model of a uniform built-up compression member applies when

1. the lacings or battenings consist of equal modules with parallel chords
2. the minimum numbers of modules in a column is three.

NOTE This assumption allows the structure to be regular and smearing the discrete structure to a continuum.

(3) The design procedure is applicable to built-up members with lacings in two directions, see Figure 6.5.

(4) The chords may be solid members or may themselves be laced or battened in the perpendicular plane.

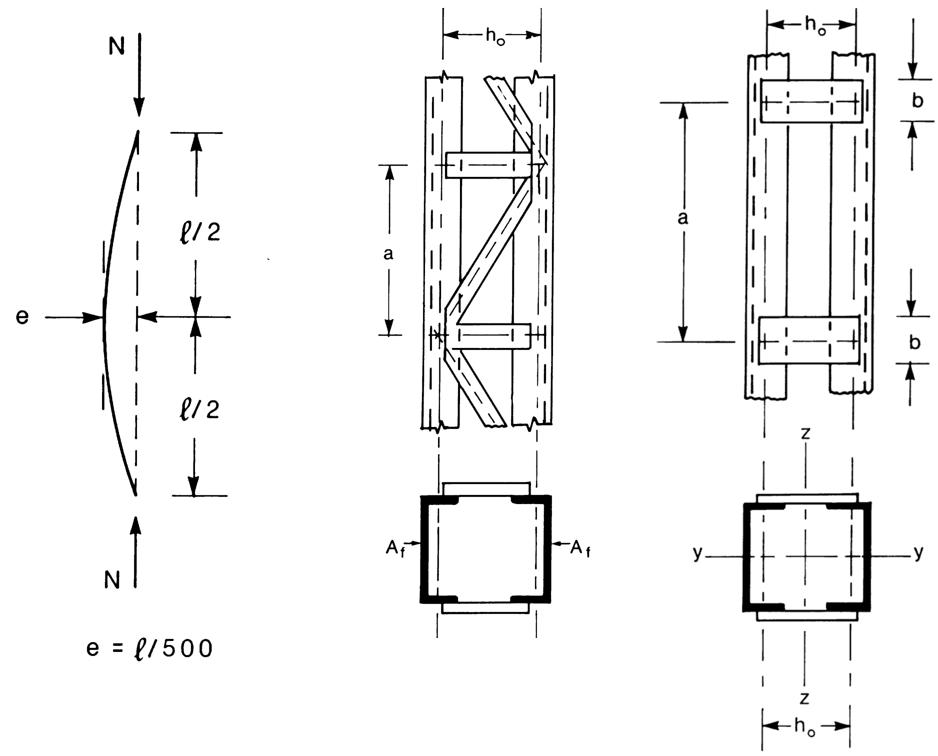


Figure 6.4: Uniform built-up columns with lacings and battenings

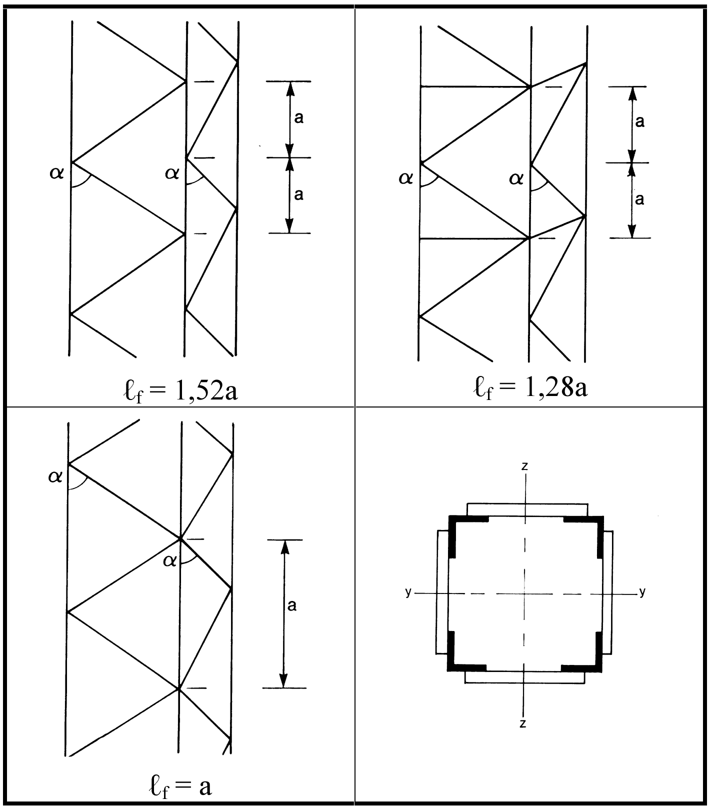


Figure 6.5: Lacings in two directions and buckling length ℓ_f of chords

(5) Checks should be performed for chords using the design chord forces $N_{f,Ed}$ from compression forces N_{Ed} and moments $M_{f,Ed}$ at mid span of the built-up member.

(6) For a member with two similar chords the design force $N_{f,Ed}$ may be determined from:

$$N_{f,Ed} = 0{,}5N_{Ed} + \frac{M_{f,Ed} h_0 A_f}{2I_{eff}} \tag{6.54}$$

where $M_{f,Ed} = \dfrac{N_{Ed} e_0 + M_{Ed}}{1 - \dfrac{N_{Ed}}{N_{cr}} - \dfrac{N_{Ed}}{S_v}}$

$N_{cr} = \dfrac{\pi^2 E I_{eff}}{\ell^2}$ is the effective critical force of the built-up member

N_{Ed} is the design value of the compression force to the built-up member

M_{Ed} is the design value of the moment at mid-length of the built-up member

h_0 is the distance between the centroids of chords

A_f is the cross-sectional area of one chord

$I_{eff} = 0{,}5h_0^2 A_f + 2\mu I_f$ is the effective second moment of area of the built-up member, see 6.4.2 and 6.4.3

S_v is the shear stiffness of the lacings or battened panel, see 6.4.2 and 6.4.3.

(7) The checks for the lacings of laced built-up members or for the frame moments and shear forces of the battened panels of battened built-up members should be performed for the end panel taking account of the shear force in the built-up member:

$$V_{Ed} = \pi \frac{M_{f,Ed}}{\ell} \tag{6.55}$$

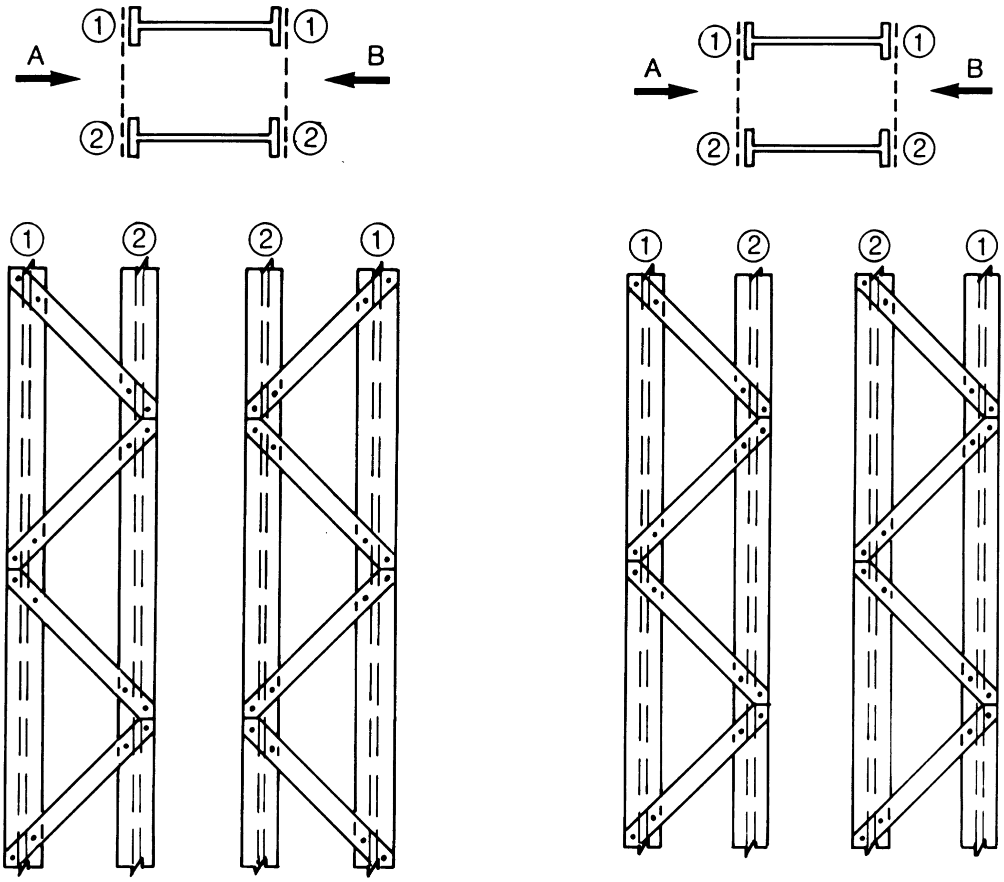
6.4.2 Laced compression members

6.4.2.1 Constructional details

(1) Single lacing systems on opposite side of the member should be corresponding systems as shown in Figure 6.6(a), arranged so that one is the shadow of the other.

(2) In case the single lacing systems on opposite sides of the main component are mutually opposed in direction as shown in Figure 6.6(b), the resulting torsional deformation of the member should be taken into account.

(3) Tie panels should be provided at the ends of lacing systems, at points where the lacing is interrupted and at joints with other members.



Lacing on face A Lacing on face B

a) Corresponding lacing system
(Recommended system)

Lacing on face A Lacing on face B

b) Mutually opposed lacing system
(Not recommended)

Figure 6.6: Single lacing system on opposite sides of main components

6.4.2.2 Resistance of components of laced compression members

(1) The chords and diagonal lacings subject to compression should be designed for buckling.

(2) For chords the buckling verification may be performed as follows:

$$\frac{N_{f,Ed}}{N_{b,Rd}} \leq 1{,}0 \qquad (6.56)$$

where $N_{f,Ed}$ is the design compression force at mid-length of the built-up member according to 6.4.1(6)

and $N_{b,Rd}$ is the design value of the buckling resistance of the chord taking the system length ℓ_f as the buckling length, see Figure 6.5.

(3) The shear stiffness of the lacings may be taken from Figure 6.7.

(4) The effective second order moment of area of laced built-up members may be taken as:

$$I_{eff} = 0{,}5 h_0^2 A_f \qquad (6.57)$$

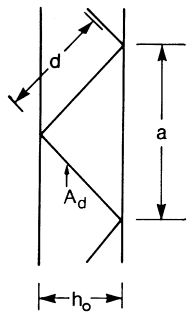
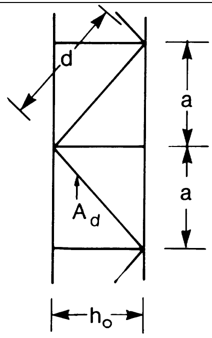
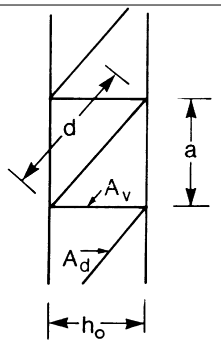
System	S_V
	$\dfrac{nEA_d a h_0^2}{2d^3}$
	$\dfrac{nEA_d a h_0^2}{d^3}$
	$\dfrac{nEA_d a h_0^2}{d^3\left[1+\dfrac{A_d h_0^3}{A_V d^3}\right]}$
n is the number of planes of lacings A_d and A_f refer to single plane	

Figure 6.7: Shear stiffness of lacings of built-up members

6.4.3 Components of battened compression members

6.4.3.1 Constructional details

(1) Battens should be supplied at each end of a member.

(2) Where parallel planes of battens are supplied, the battens in each plane should be arranged opposite each other.

(3) Battens should also be supplied at intermediate points where loads are applied or lateral restraint is supplied.

6.4.3.2 Resistance of components of battened compression members

(1) The chords and the battens and their joints to the chords should be checked for the moments and forces in an end panel and at mid-span as indicated in Figure 6.8.

NOTE For simplicity the maximum chord forces $N_{f,Ed}$ may be combined with the maximum shear force V_{Ed}.

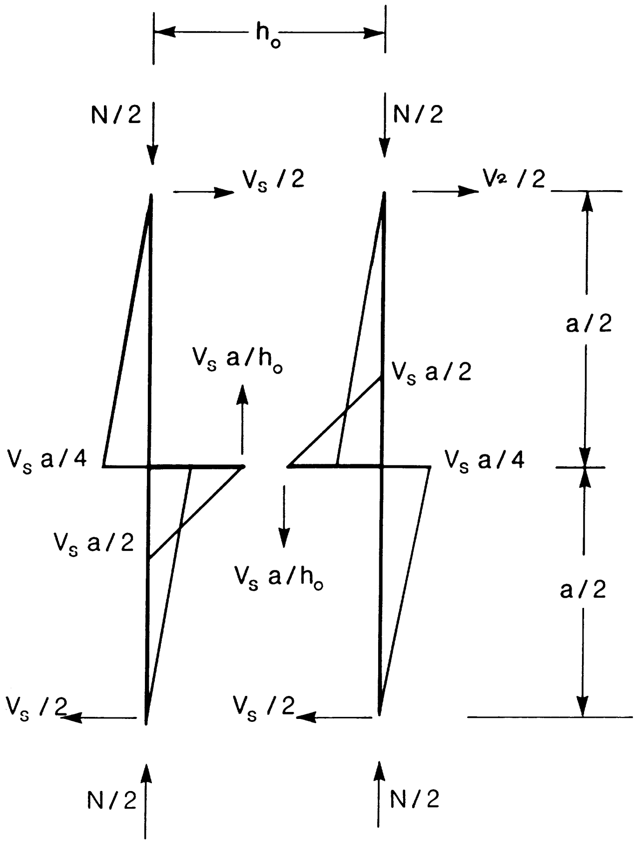


Figure 6.8: Moments and forces in an end panel of a battened built-up member

(2) The shear stiffness S_V may be taken from Table 6.9.

(3) The effective second moments of area of battened built-up members may be taken as:

$$I_{eff} = 0{,}5h_0^2 A_f + 2\mu I_f \tag{6.58}$$

where I_f = in plane second moment of area of one chord

I_b = in plane second moment of area of one batten

μ = efficiency factor from Table 6.8

Table 6.8: Efficiency factor μ

criterion	efficiency factor
$\lambda = \frac{\ell}{i_0}$; $i_0 = \sqrt{\frac{I_1}{2A_f}}$ $I_1 = 0{,}5h_0^2 A_f + 2I_f$	μ
$\lambda \geq 150$	0
$75 < \lambda < 150$	$\mu = 2 - \frac{\lambda}{75}$
$\lambda \leq 75$	1,0

Table 6.9: Shear stiffness S_V

criterion	S_V
$\frac{nI_b}{h_0} \geq 10\frac{I_f}{a}$	$S_v = \frac{2\pi^2 EI_f}{a^2}$
$\frac{nI_b}{h_0} < 10\frac{I_f}{a}$	$S_v = \frac{24EI_f}{a^2\left[1 + \frac{2I_f}{nI_b}\frac{h_0}{a}\right]} \leq \frac{2\pi^2 EI_f}{a^2}$

6.4.4 Closely spaced built-up members

(1) Built-up compression members with main components in contact or closely spaced and connected through packing plates, see Figure 6.9, or star battened angle members connected by pairs of battens in two perpendicular planes, see Figure 6.10 may be checked for buckling as a single integral member ignoring the effect of shear stiffness ($S_V = \infty$), when the conditions in Table 6.10 are met.

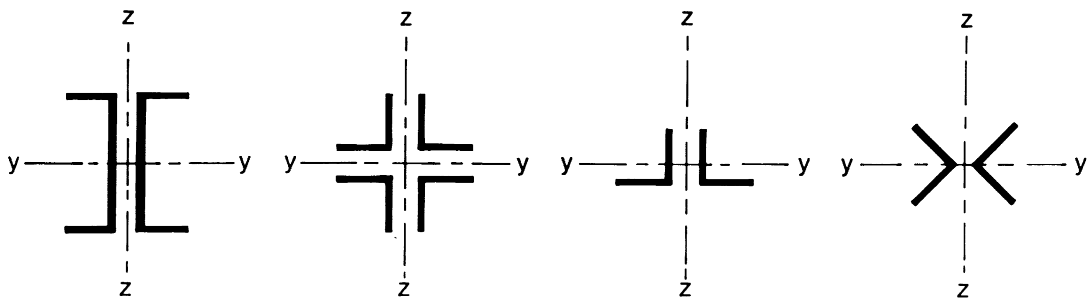


Figure 6.9: Closely spaced built-up members

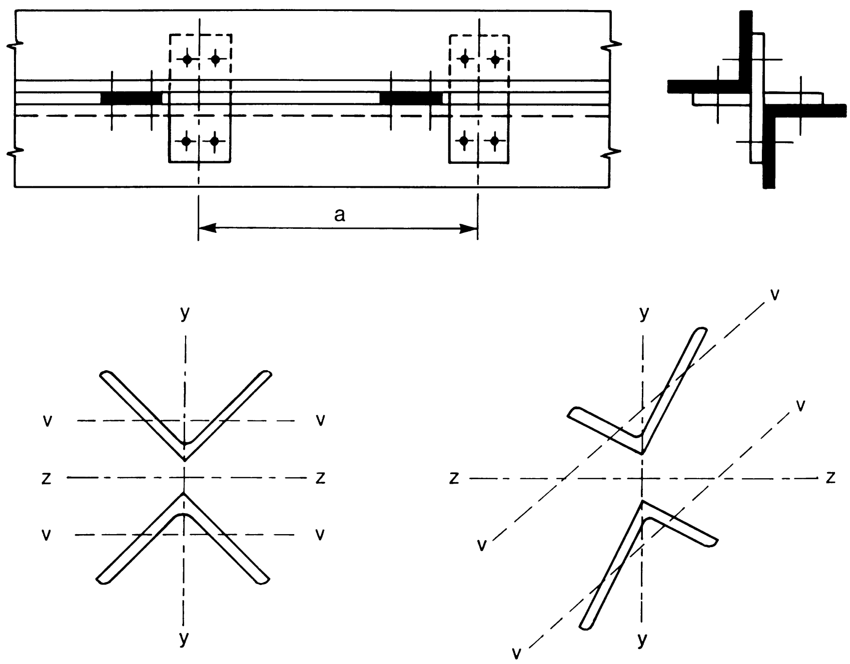


Figure 6.10: Star-battened angle members

Table 6.10: Maximum spacings for battens for closely spaced built-up or star battened angle members

	Maximum spacing between battens
Closely spaced built-up members according to Figure 6.9	15 i_{min}
Star battened angle members	70 i_{min}

(2) The shear forces to be transmitted by the battens may be determined from 6.4.3.2(3).

(3) In the case of unequal-leg angles, see Figure 6.10, buckling about the y-y axis may be verified with:

$$i_y = \frac{i_0}{1,15} \tag{6.59}$$

where i_0 is the minimum radius of gyration of the built-up member.

7 Serviceability limit states

(1) A steel structure shall be designed and constructed such that all relevant serviceability limit states are satisfied.

(2) The basic requirements for serviceability limit states are given in 3.3 of prEN 1990.

(3) Any serviceability limit state and the accociated loading and analysis model should be specified in the project documents.

(4) When plastic global analysis is used for the ultimate limit state, plastic redistribution of forces and moments at the serviceability limit state may be allowed. If so, the effects have to be considered.

8 Fasteners, welds, connections and joints

(1) For the design of fasteners, weld, connections and joints see EN 1993-1-8.

Annex A [informative] – Recommendations for alternative interaction factors in 6.3.4(3) and 6.3.4(5)

A.1 General

(1) In this annex alternative methods A and B are recommended for determining alternative values of the interaction factors k_{yy} , k_{yz} , k_{zy} and k_{zz} in 6.3.4(3) and 6.3.4(5).

(2) Method A refers to the values in Table 6.6 and gives approximate modifications of these values that may be applied for class 1 and class 2 cross section when plastic resistances are used.

(3) Method B gives approximate values for k_{yy} , k_{yz} , k_{zy} and k_{zz} that may be applied instead of the values in Table 6.6 for class 3 and class 4 cross sections and also for class 1 and class 2 cross sections.

NOTE Any of these alternative methods given in this annex makes the interaction dependant on National Determined Parameters.

A.2 Method A: Modification of the interaction factors for k_{yy} , k_{yz} , k_{zy} and k_{zz} for class 1 and class 2 cross sections

(1) For class 1 and class 2 cross sections the interaction factors for k_{yy} , k_{yz} , k_{zy} and k_{zz} according to Table 6.6 may be modified to $k_{yy,mod}$, $k_{yz,mod}$, $k_{zy,mod}$ and $k_{zz,mod}$ according to Table A.2.1.

(2) In Table A.2.1 $\bar{\lambda}_{LT,0} = 0$ applies for members not susceptible to torsional deformations.

Table A.2.1: Modification of interaction factors

$k_{yy,mod}$	$\dfrac{k_{yy}}{1+\left(w_y-1\right)\left[\left(2-\dfrac{1,6}{w_y}C_{m.y}^{2}\,\bar{\lambda}_{max}-\dfrac{1,6}{w_y}C_{m.y}^{2}\,\bar{\lambda}_{max}^{2}\right)n_{pl}-b_{LT}\right]}\le k_{yy}\dfrac{W_{pl.y}}{W_{el.y}}$
$k_{yz,mod}$	$\dfrac{0,6\sqrt{\dfrac{w_z}{w_y}}\,k_{yz}}{1+\left(w_z-1\right)\left[\left(2-14\dfrac{C_{m.z}^{2}\,\bar{\lambda}_{max}^{2}}{w_z^{5}}\right)n_{pl}-c_{LT}\right]}\le k_{yz}\dfrac{1}{0,6}\sqrt{\dfrac{w_y}{w_z}}\dfrac{W_{pl.z}}{W_{el.z}}$
$k_{zy,mod}$	$\dfrac{0,6\sqrt{\dfrac{w_y}{w_z}}\,k_{zy}}{1+\left(w_y-1\right)\left[\left(2-14\dfrac{C_{m.y}^{2}\,\bar{\lambda}_{max}^{2}}{w_y^{5}}\right)n_{pl}-d_{LT}\right]}\le k_{zy}\dfrac{1}{0,6}\sqrt{\dfrac{w_z}{w_y}}\dfrac{W_{pl.y}}{W_{el.y}}$
$k_{zz,mod}$	$\dfrac{k_{zz}}{1+\left(w_z-1\right)\left[\left(2-\dfrac{1,6}{w_z}C_{m.z}^{2}\,\bar{\lambda}_{max}-\dfrac{1,6}{w_z}C_{m.z}^{2}\,\bar{\lambda}_{max}^{2}\right)n_{pl}-e_{LT}\right]}\le k_{zz}\dfrac{W_{pl.z}}{W_{el.z}}$

where

$$w_y=\frac{W_{pl,y}}{W_{el,y}}\le 1,5$$

$$w_z=\frac{W_{pl,z}}{W_{el,z}}\le 1,5$$

$$n_{pl}=\frac{N_{Ed}}{N_{c,Rk}/\gamma_{M1}}$$

$$\bar{\lambda}_{max}=\max\begin{cases}\bar{\lambda}_y\\ \bar{\lambda}_z\end{cases}$$

$\bar{\lambda}_{LT.0}$ = non-dimensional slenderness for lateral-torsional buckling due to uniform bending moment, i.e. ψ_y =1,0 in Table 6.7

$$b_{LT}=0,5\,a_{LT}\,\bar{\lambda}_{LT0}^{2}\,\frac{M_{y.Ed}}{\chi_{LT}\,M_{pl.y.Rd}}\,\frac{M_{z.Ed}}{M_{pl.z.Rd}}$$

$$c_{LT}=10\,a_{LT}\,\frac{\bar{\lambda}_{LT0}^{2}}{5+\bar{\lambda}_z^{4}}\,\frac{M_{y.Ed}}{C_{m.y}\,\chi_{LT}\,M_{pl.y.Rd}}$$

$$d_{LT}=2\,a_{LT}\,\frac{\bar{\lambda}_{LT0}}{0,1+\bar{\lambda}_z^{4}}\,\frac{M_{y.Ed}}{C_{m.y}\,\chi_{LT}\,M_{pl.y.Rd}}\,\frac{M_{z.Ed}}{C_{m.z}\,M_{pl.z.Rd}}$$

$$e_{LT}=1.7\,a_{LT}\,\frac{\bar{\lambda}_{LT0}}{0,1+\bar{\lambda}_z^{4}}\,\frac{M_{y.Ed}}{C_{m.y}\,\chi_{LT}\,M_{pl.y.Rd}}$$

A.3 Method B: Alternative interaction factors k_{yy} , k_{yz} , k_{zy} and k_{zz}

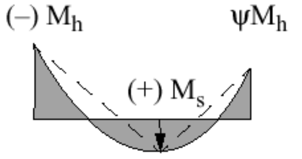
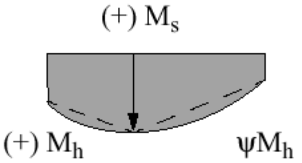
(1) For members not susceptible to torsional deformations with class 1, class 2, class 3 and class 4 cross section alternative interaction factors k_{yy} , k_{yz} , k_{zy} and k_{zz} according to 6.3.4(3) may be determined from Table A.3.1.

Table A.3.1: Interaction factors for members not susceptible to torsional deformations

Cross sectional class		I-sections	RHS-sections
1 and 2	k_{yy}	$1+\left(\bar{\lambda}_y-0{,}2\right)\dfrac{N_{Ed}}{\chi_y N_{c,Rk}/\gamma_{M1}}$ $\leq 1+0{,}8\dfrac{N_{Ed}}{\chi_y N_{c,Rk}/\gamma_{M1}}$	see I-sections
	k_{yz}	$0{,}6\ k_{zz}$	$0{,}6\ k_{zz}$
	k_{zy}	$0{,}6\ k_{yy}$	see I-sections
	k_{zz}	$1+\left(2\bar{\lambda}_z-0{,}6\right)\dfrac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$ $\leq 1+1{,}4\dfrac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$	$1+\left(\bar{\lambda}_z-0{,}2\right)\dfrac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$ $\leq 1+0{,}8\dfrac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$
3 and 4	k_{yy}	$1+0{,}6\bar{\lambda}_y\dfrac{N_{Ed}}{\chi_y N_{c,Rk}/\gamma_{M1}}$ $\leq 1+0{,}6\dfrac{N_{Ed}}{\chi_y N_{c,Rk}/\gamma_{M1}}$	see I-sections
	k_{yz}	$0{,}8\ k_{zz}$	see I-sections
	k_{zy}	k_{yy}	see I-sections
	k_{zz}	$1+0{,}6\bar{\lambda}_z\dfrac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$ $\leq 1+0{,}6\dfrac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$	see I-sections

(2) The equivalent uniform moment factors C_{my} , C_{mz} and C_{mLT} may be taken from Table A.3.2.

Table A.3.2: Equivalent uniform moment factors C_{my} , C_{mz} and C_{mLT}

Moment diagram	range	C_{my} , C_{mz} and C_{mLT}	
		uniform loading	concentrated load
	$0 \le \alpha_s \le 1$ $-1 \le \psi \le 1$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$
$\alpha_s = M_s/M_h$	$-1 \le \alpha_s \le 0$ $0 \le \psi \le 1$ $\psi = -1$	$0{,}1 - 0{,}8\alpha_s \ge 0{,}4$ $0{,}2 - 0{,}8\alpha_s \ge 0{,}4$	$-0{,}8\alpha_s \ge 0{,}4$ $0{,}2 - 0{,}8\alpha_s \ge 0{,}4$
	$0 \le \alpha_h \le 1$ $-1 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
$\alpha_h = M_h/M_s$	$-1 \le \alpha_h \le 0$ $0 \le \psi \le 1$ $\psi = -1$	$0{,}95 + 0{,}05\alpha_h$ $0{,}95 - 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$ $0{,}90 - 0{,}10\alpha_h$
	$-1 \le \psi \le 1$	$0{,}6 + 0{,}6\psi \ge 0{,}4$	

(3) For members susceptible to torsional deformations (I-sections) the interaction factors may be taken from Table A.3.3, whereby $C_{mLT} = C_{my}$.

Table A.3.3: Interaction factors for members susceptible to torsional deformations

Cross sectional class		
1 and 2	k_{yy}	k_{yy}
	k_{yz}	$1-\frac{0{,}1\bar{\lambda}_z}{(C_{m,LT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}} \ge 1-\frac{0{,}1}{(C_{m,LT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$
3 and 4	k_{yy}	k_{yy}
	k_{yz}	$1-\frac{0{,}05\bar{\lambda}_z}{(C_{m,LT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}} \ge 1-\frac{0{,}05}{(C_{m,LT}-0{,}25)}\frac{N_{Ed}}{\chi_z N_{c,Rk}/\gamma_{M1}}$