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Part 3 : Buildings

Calcul des structures en acier

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Ingènierie du bâtiment

Bemessung und Konstruktion von Stahlbauten

Teil 3 :

Hochbau

CEN

European Committee for Standardisation
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Content

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Foreword

This European Standard EN 1993-3, Design of Steel Structures : Buildings, 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 1993-2 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No existing European Standard is superseded.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 1993-3

EN 1993-3 is the third part of seven parts of EN 1993 – Design of Steel Structures – and describes the principles and application rules for the safety and serviceability and durability of steel structures for buildings.

EN 1993-3 gives design rules in supplement to the generic rules in EN 1993-1.

EN 1993-3 is intended to be used with Eurocodes EN 1990 – Basis of design, EN 1991 – Actions on structures and the parts 1 of EN 1992 to EN 1998 when steel structures or steel components for buildings are referred to.

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

EN 1993-3 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 1993-3

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 1993-3 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 1993-3 through clauses:

- 2.1.3.2(2)
- 2.1.3.4(3)
- 6.3.2(2)
- 6.3.2(3)
- 7.2.3(1)

1 General

1.1 Scope

1.1.1 Scope of Eurocode 3

(1) See 1.1.1 of EN 1993-1-1.

1.1.2 Scope of Part 3 of Eurocode 3

(1) This Part 3 of EN 1993 gives a general basis for the structural design of steel buildings, steel parts of composite buildings or buildings that are mainly of other construction materials and also steel temporary works in buildings. It gives provisions that supplement, modify or supersede the equivalent provisions given in the various parts of EN 1993-1.

(2) This Part 3 also gives detailed application rules that are mainly applicable to commonly used types of buildings. Where the applicability of these rules is limited, for practical reason or due to simplifications, their use and any limits of applicability are explained in the text.

(3) Provisions for the structural fire design, the design of cold formed thin gauge members and sheeting, the design with stainless steels and with tensile elements are included in EN 1993-1 and therefore not dealt with in this part.

(4) Provisions for composite components are covered in EN 1994-1.

(5) The design of steel bearing piles and steel sheet pile walls is covered in EN 1993-5.

(6) Provisions for the design of crane supporting structures are included in EN 1993-6.

(7) This standard is concerned only with provisions for resistance, serviceability and durability of bridge structures. Other aspects of design are not considered.

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

(9) For the execution of steel buildings, reference should be made to EN xxx[5].

(10) EN 1993-3 does not cover the special requirements of seismic design. Reference shall be made to the requirements given in EN 1998, which complements and modifies the rules of EN 1993-3 specifically for this purpose.

(11) The following subjects are dealt with in EN 1993-3:

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: Connections

[5] EN xxx is the conversion of ENV 1090

(12) Section 1 to 2 provide additional clauses to those given in EN 1990 “Basis of structural design”.

(13) Section 3 deals with material properties of products used in building design.

(14) Section 4 gives general rules for durability.

(15) Section 5 refers to the structural analysis of building structures.

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

(17) Section 7 gives rules for serviceability.

(18) Section 8 refers to connections used in building.

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.

EN xxx[5] Execution of steel structures:

1.3 Assumptions

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

1.4 Distinction between principles and application rules

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

1.5 Definitions

(1) For the purposes of this Part 3 of EN 1993, in addition to the definitions given in EN 1990 and EN 1993-1, the following definitions apply:

Draft note: ... to be inserted later.

1.6 Symbols

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

Draft note: ... to be inserted later.

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

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

2.1.2 Reliability management

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

2.1.3 Design working life, durability and robustness

2.1.3.1 General

(1) See 2.1.3 of EN 1993-1-1.

2.1.3.2 Design working life

(1) The design working life should be taken as the period for which a building is required to be used for its intended purpose.

(2) The intended design working life of a permanent building should be taken as 50 years unless otherwise specified.

NOTE For temporary buildings the design working life should be defined in the National Annex.

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

2.1.3.3 Durability

(1) To ensure durability, buildings and their components should either be designed for the environmental actions, fatigue if relevant, and accidental actions that are expected during the design working life, or else protected from them.

(2) Where a building includes components that need to be replaceable (e.g. bearings in zones of soil settlement), the possibility of their safe replacement should be verified as a transient design situation, taking into account (as far as possible) the need to minimise interruption to the use of the building.

2.1.3.4 Robustness and structural integrity

(1) Buildings should be designed to tolerate specified damages.

(2) The design should ensure that when damage due to accidental actions occurs, the remaining structure can sustain at least the accidental load combination.

(3) The effects of deterioration of material, corrosion or fatigue where relevant should be taken into account by appropriate choice of material, see EN 1993-1-10, and details, see EN 1993-1-9, or structural redundancy and corrosion protection system.

NOTE The National Annex may give more detailed provisions for structural redundancy.

2.2 Principles of limit state design

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

2.3 Basic variables

(1) See 2.3 of EN 1993-1-1.

2.4 Verification by the partial factor method

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

2.5 Design assisted by testing

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

3 Materials

3.1 General

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

3.2 Structural steel

(1) See 3.2 of EN 1993-1-1.

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.

3.4 Other items

(1) Any semi-finished or finished structural product used in the structural design of buildings should comply with the relevant technical specification.

4 Durability

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

(2) For building structures no fatigue assessment is normally required except as follows:

a) Members supporting lifting appliances or rolling loads

b) Members subject to repeated stress cycles from vibrating machinery

c) Members subject to wind-induced vibrations

d) Members subject to crowd-induced oscillations

(3) Buildings with an internal structure protected by a facade normally don't need any corrosion protection.

Draft note: This is the case for an internal relative humidity less or equal to 80%.

5 Structural analysis

5.1 Structural modelling for analysis

5.1.1 Structural modelling and basic assumptions

(1) See 5.1.1 of EN 1993-1-1.

(2) For the structural modelling and basic assumptions for components of buildings see also EN 1993-1-5 and EN 1993-1-11.

5.1.2 Joint modelling

(1) See 5.1.2 of EN 1993-1-1.

5.1.3 Ground structure interaction

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

5.2 Structural stability

5.2.1 Effects of deformed geometry of the structure

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

(2) Beam-and-column type plane frames in buildings may be checked with first order theory if the following criterion is satisfied:

$$\left(\frac{\delta_H}{h}\right) \cdot \left(\frac{N_{Ed}}{H_{Ed}}\right) \leq 0{,}1 \qquad (5.1)$$

where δ_H is the horizontal displacement belonging to H according to first order theory at the top of the storey, relative to the bottom of the storey

h is the storey height

H_{Ed} is a reference horizontal reaction at the bottom of the storey

N_{Ed} is the total vertical reaction at the bottom of the storey.

5.2.2 Method of analysis

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

(2) As an alternative to (1) the effects of flexural stability of a building structure may be verified by a member check according to 6.3 based on buckling length values taken from a global structural analysis, including consideration of elastic restraint of members and joints and the actual distribution of the design compression forces. In this case internal forces are calculated according to first order theory without considering imperfections. Special considerations should be given to the actual deformation and reaction forces for the verification of the stabilising system and members.

NOTE In general structural stability may be achieved by bracing.

5.3 Imperfections

5.3.1 Basis

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

5.3.2 Imperfections for global analysis

(1) See 5.3.2 of EN 1993-1-1.

(2) For building-frames the effect of imperfections should be allowed for in frame analysis by means of an equivalent imperfection in the form of an initial sway imperfection ϕ, see Figure 5.1, determined from:

$$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m \tag{5.2}$$

where ϕ_0 is the basic value: $\phi_0 = 1/200$

α_h is the reduction factor for height h applicable to continuous columns: $\alpha_h = \dfrac{2}{\sqrt{h}} \quad \begin{matrix} \leq 1 \\ \geq 2/3 \end{matrix}$

α_m is the reduction factor for the number of columns in a row: $\alpha_m = \sqrt{0{,}5\left(1 + \dfrac{1}{m}\right)}$

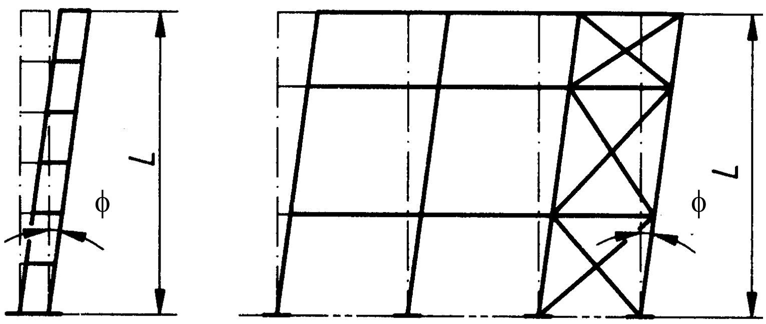


Figure 5.1: Equivalent sway imperfections for buildings

(3) Columns which carry a vertical load N_{Ed} of less than 50% of the average value of the vertical load per column in the plane considered, shall not be included in m.

(4) These initial sway imperfections apply in all horizontal directions, but need only be considered in one direction at a time.

(5) The possible torsional effects on the structure on anti-symmetric sways, on two opposite faces, should also be considered, see Figure 5.2.

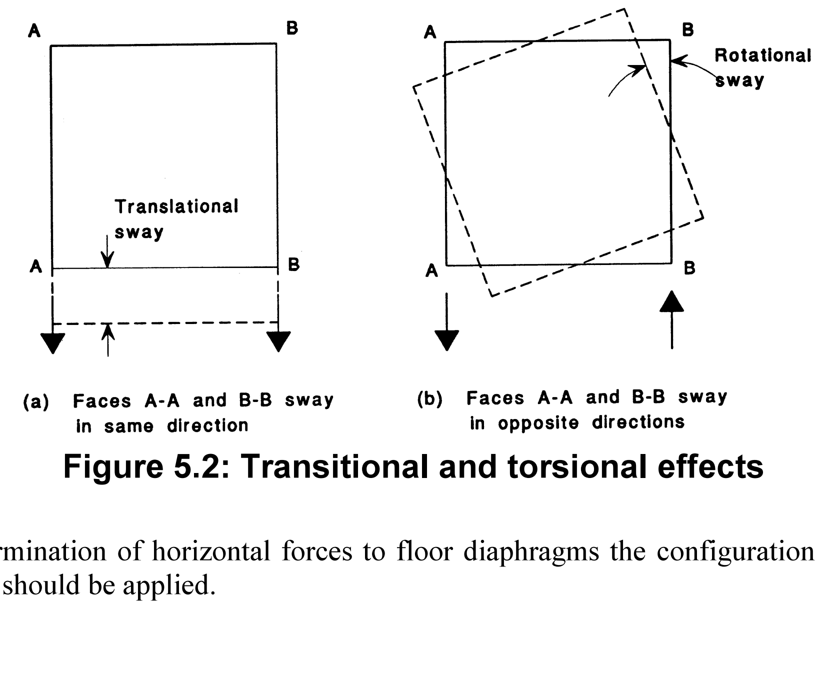


Figure 5.2: Transitional and torsional effects

(6) For the determination of horizontal forces to floor diaphragms the configuration of imperfections as given in Figure 5.3 should be applied.

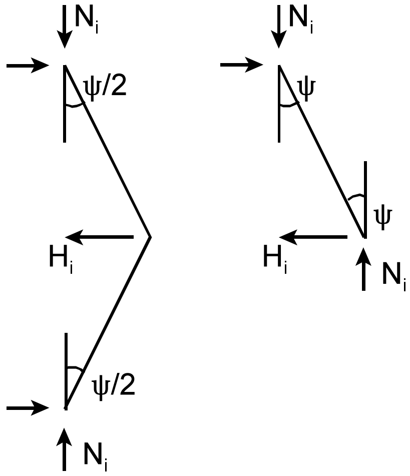


Figure 5.3: Configuration of imperfections for floor diaphragms

(7) If more convenient, the initial sway imperfection may be replaced by a close system of equivalent horizontal forces, introduced for each column, see Figure 5.4.

(8) In multiple beam-and-column building frames, these equivalent forces should be applied at each floor and roof level and should be proportionate to the design vertical loads applied to the structure at that level for the load case under consideration.

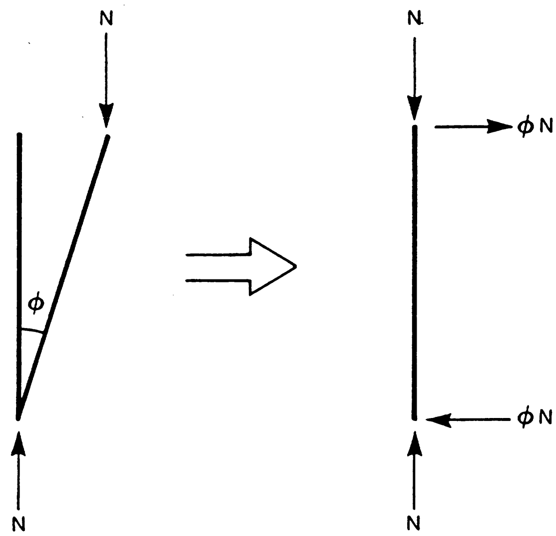


Figure 5.4: Replacement of initial sway imperfections by equivalent horizontal forces

5.3.3 Imperfection for analysis of bracing systems

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

5.3.4 Member imperfections

(1) See 5.3.4 of EN 1993-1-1.

(2) For building frames with sway imperfections according to 5.3.2(2) also member imperfections according to 5.3.2 should be taken into account in addition. They may be neglected if the following criterion applies:

$$\bar{\lambda} \leq 0,5 \cdot \sqrt{\frac{A \cdot f_y}{N_{Ed}}} \qquad (5.3)$$

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

and $\bar{\lambda}$ is the in-plane non-dimensional slenderness calculated using a buckling length equal to the system length.

5.4 Calculation of action effects

(1) See 5.4 of EN 1993-1-1.

(2) As a simplified method for the calculation of action effects following a first-order analysis, see EN 1993-1-1 5.2.1, the plastic global analysis may be replaced by an elastic global analysis and modifying the calculated elastic bending moments by redistributing up to 15 % of the peak calculated moment in any member, provided, that:

a) the internal forces and moments in the frame remain in equilibrium with the applied loads, and

b) all the members in which the moments are reduced have Class 1 or Class 2 cross-sections (see EN 1993-1-1, 5.5).

5.5 Classification of cross sections

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

6 Ultimate limit states

6.1 General

(1) See 6.1 of EN 1993-1-1.

6.2 Resistance of cross-sections

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

6.3 Buckling resistance of members

6.3.1 Compression members

(1) See 6.3.1 of EN 1993-1-1.

NOTE For buckling of components of building structures see Annex B.

6.3.2 Lateral-torsional buckling of beams

(1) See 6.3.2 of EN 1993-1-1.

(2) Members in buildings with lateral restraint to the compression flange are not susceptible to lateral-torsional buckling if the stable length of segment L_c corresponding to the slenderness $\bar{\lambda}_f$ of the equivalent compression flange does not exceed:

$$\bar{\lambda}_f = \frac{k_c L_c}{i_{f,z} \lambda_1} \leq \bar{\lambda}_{c0} \frac{M_{c,Rd}}{M_{y,Sd}} \qquad (6.1)$$

where $M_{y,Sd}$ is the maximum design value of the bending moment within the restraint spacing

$$M_{c,Rd} = W_y \frac{f_y}{\gamma_{M1}}$$

W_y is the appropriate section modulus of the compression flange

k_c is a slenderness correction factor for moment distribution between restraints, see Table 6.1

$i_{f,z}$ is the radius of gyration of the compression flange about the minor axis the section including 1/3 of the compressed part of the web area

$\bar{\lambda}_{c0}$ is the slenderness limit for the equivalent compression element

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

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

NOTE 1 $i_{f,z}$ may be taken as the radius of gyration about the minor axis the section, i_z .

$$i_{f,z} = \sqrt{\frac{I_{eff,f}}{A_{eff,f} + \frac{1}{3}A_{eff,w,c}}} \quad \text{for Class 4 cross-sections}$$

where $I_{eff,f}$ is the effective second moment of area of the compression flange about the minor axis of the section

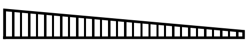
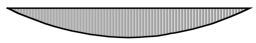
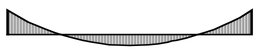
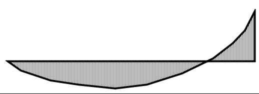
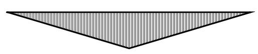
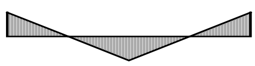
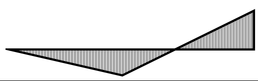
$A_{eff,f}$ is the effective areas of the flange

$A_{eff,w,c}$ is the effective areas of the compressed web

and $A_{eff,w,c} < A_{eff,w}$

NOTE 2 The slenderness limit $\bar{\lambda}_{c0}$ may be given in the National Annex. A limit value $\bar{\lambda}_{c0} = 0{,}3$ is recommended.

Table 6.1: Correction factors for class 1, 2 and 3 sections

Moment distribution	k_c
 $\psi = 1$	1,0
 $-1 \le \psi \le 1$	$\frac{1}{1{,}33 - 0{,}33\psi}$
	0,94
	0,90
	0,91
	0,86
	0,77
	0,82
NOTE For Class 4 cross-sections k_c =1,0.	

(3) If the slenderness of the compression flange $\bar{\lambda}_f$ exceeds the limit given in (2), the design buckling resistance moment may be taken as:

$$M_{b,Rd} = k_{f\ell}\chi\, M_{c,Rd} \quad \text{but} \quad M_{b.Rd} \leq M_{c.Rd} \tag{6.2}$$

where χ is the reduction factor of the equivalent compression member

$k_{f\ell}$ is the magnification factor accounting for the safesidedness of the equivalent compression element

NOTE The magnification factor may be given in the National Annex. A value $k_{f\ell} = 1{,}10$ is recommended.

(4) The buckling curves to be used in (3) should be taken as follows:

curve c for rolled sections

curve d for welded sections provided that: $\dfrac{h}{t_f} \leq 44\varepsilon$

where h is the overall depth of the cross-section

t is the thickness of the compression flange

NOTE For lateral torsional buckling of components of building structures see Annex B.

6.3.3 Lateral torsional buckling of frames

6.3.3.1 General method

(1) See 6.3.3.1 of EN 1993-1-1.

6.3.3.2 Lateral torsional buckling of portal frames with plastic design

6.3.3.2.1 General

(1) Portal frames may be designed with plastic analysis provided lateral torsional buckling of the frame is prevented by the following means:

a) restraints at "rotated" plastic hinges

b) verification of stable length of segment between such restraints

(2) Where it can be demonstrated that, under all ultimate state load combinations, the plastic hinge is "non-rotated", because under that load combination it is the last hinge to form or it is not yet fully formed, no restraints are necessary to obtain stable length´s of segment.

6.3.3.2.2 Restraints at rotated plastic hinges

(1) Under all ultimate limit state load combinations, both flanges should have lateral restraint at each rotated plastic hinge location, designed to resist a force equal to 2.5% of the force in the compression flange. Where it is not practicable to provide such restraint directly at the hinge location, it should be provided within a distance d/2 along the length of the member, where d is its overall depth at the plastic hinge location.

6.3.3.2.3 Verification of stable length of segment

(1) The lateral torsional buckling verification of uniform segments to determine the stable length may be performed according to 6.3.2.

(2) Where a plastic hinge location occurs immediately adjacent to one end of a haunch, the tapered segment need not be treated as a segment adjacent to a plastic hinge location if the following criteria are satisfied:

1. For three flange haunches:
   a) the restraint at the plastic hinge location should be within a distance d/2 along the length of the tapered segment, no the uniform segment
   b) the haunch remains elastic throughout its length
2. For two flange haunches
   a) the moment at the lateral restraint does not exceed 85% of the plastic moment resistance reduced to allow for an axial load
   b) the length L_y to the adjacent lateral restraint to the compression flange does not exceed 85% of the limiting length

NOTE A lateral torsional buckling verification of tapered segments to determine the stable length may be performed according to Annex B.

6.3.4 Bending and axial compression

(1) See 6.3.4 of EN 1993-1-1.

6.4 Built-up compression members

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

6.5 Buckling of plates

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

7 Serviceability limit states

7.1 General

(1) See 7 of EN 1993-1-1.

7.2 Serviceability limit states for buildings

7.2.1 General

(1) With reference to EN 1990 – Annex A 1.4.3 – Figure A.11 the limits for vertical deflections given in Table 7.1 are recommended.

Table 7.1: Example for limiting design values for deflections as a function of span L, or twice the length of a cantilever

Serviceability requirement	Characteristic combination of actions
Deflection	$\delta_{max} - \delta_c$
Irreversible limit states - Limit deformations to control cracking for particular elements	
Partitions: – brittle partition walls(not reinforced) – reinforced partition walls – removable partition walls	 $\leq$ L/500 $\leq$ L/350 $\leq$ L/250
Ceilings: – plastered ceiling – false ceiling	 $\leq$ L/350 $\leq$ L/250
Flooring: – rigid flooring (e.g. ceramic, tiles,...) – flexible flooring (e.g. flexible floor covering)	 $\leq$ L/500 $\leq$ L/250
Irreversible Limit States - Limit deflection to ensure drainage of water	
Roof covering: – rigid covering – flexible covering	 $\leq$ L/250 $\leq$ L/125

7.2.2 Recommendations for horizontal deflections

(1) With reference to EN 1990 – Annex A 1.4.3 – Figure A.11 the limits for horizontal deflections given in Table 7.2 are recommended.

Table 7.2: Examples for limiting design values of horizontal deflections as a function of height H of the building, storey height ΔH or span L

Serviceability requirement	**Combination of actions**
	Characteristic
Partitions	$\Delta u \leq \Delta H/500$
Appearance of the structure	$\Delta u \leq \Delta H/250$

7.2.3 Dynamic effects

(1) The vibrations of structure on which the public can walk shall be limited to avoid significant discomfort to users.

NOTE 1 The natural frequency (f) may be estimated as an approximation for simply supported beams as follows:

$$f = \frac{15{,}8}{\sqrt{\delta}} \tag{7.1}$$

where δ is the deflection in [mm]

NOTE 2 The National Annex may specify limits for vibration of floors.

8 Fasteners, welds, connections and joints

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

Annex A [informative] – Simplified provisions for the design of continuous floor beams

Draft note: To be drafted after agreement in the CEN / TC 250 Coordination Group.

Annex B [informative] – Buckling of components of building structures

B.1 Flexural buckling of columns in frames with rigid connections

(1) The buckling length L_{cr} of a column in a non-sway mode may be obtained from Figure B.1.

(2) The buckling length L_{cr} of a column in a sway mode may be obtained from Figure B.2.

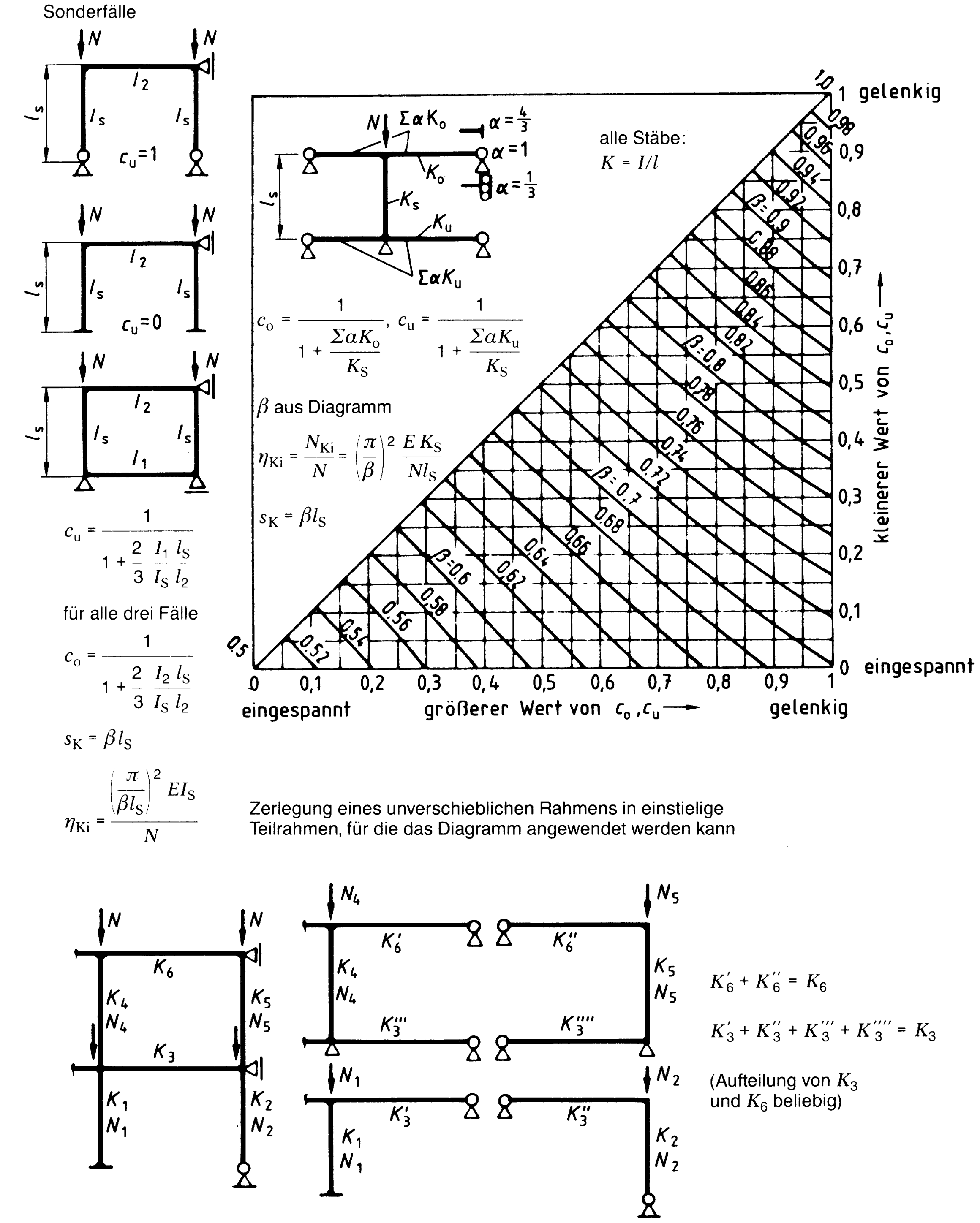


Bild 27. Diagramm zur Bestimmung des Verzweigungslastfaktors η_{Ki} und der Knicklängen s_K für Stiele unverschieblicher Rahmen mit $\varepsilon_{Riegel} \leq 0{,}3$

Figure B.1: Buckling length ratio L_{cr} / L for a column in a non-sway mode

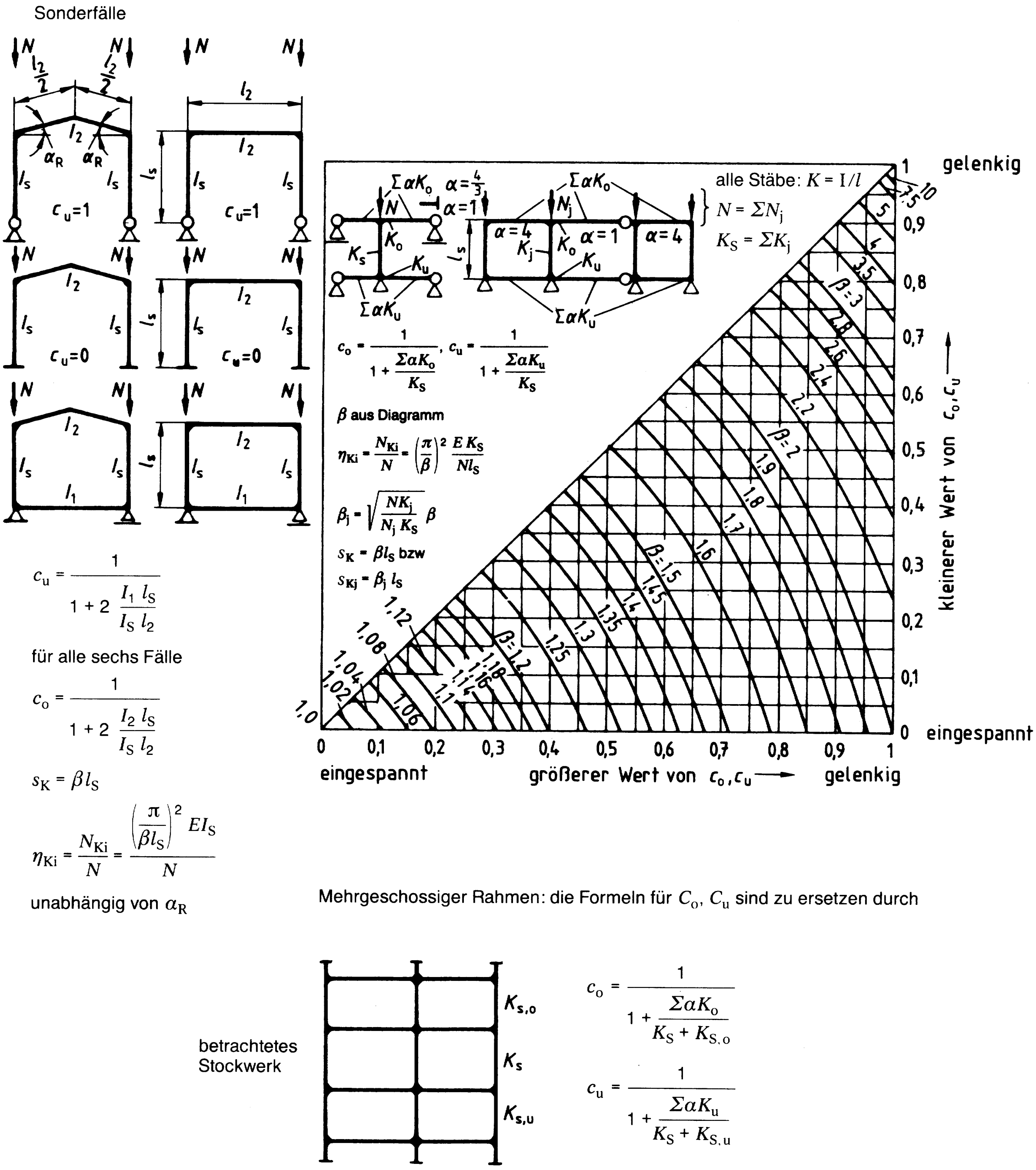


Bild 29. Diagramm zur Bestimmung des Verzweigungslastfaktors η_{Ki} und der Knicklänge s_K für Stiele verschieblicher Rahmen mit $\varepsilon_{Riegel} \leq 0{,}3$

Figure B.2: Buckling length ratio L_{cr} / L for a column in a sway mode

B.2 Flexural buckling of members in triangulated and lattice structures

B.2.1 General

(1) For chord members generally and for out-of-plane buckling of web members, the buckling length L_{cr} shall be taken as equal to the system length L, unless a smaller value is justified by analysis.

(2) Web members may be designed fir in-plane buckling using a buckling length smaller than the system length, provided the chords supply appropriate end restraint and the end connections supply appropriate fixity (at least 2 bolts if bolted).

(3) Under these conditions, in normal triangulated structures the buckling length L_{cr} of web members for in-plane buckling may be taken as 0,9L, except for angle sections, see B.2.2.

B.2.2 Angles as web members

(1) Provided that the chords supply appropriate end restraint to web members made of angles and the end connections of such web members supply appropriate fixity (at least two bolts if bolted), the eccentricities may be neglected and end fixities allowed for in the design of angles as web members in compression. The effective slenderness ratio $\bar{\lambda}_{eff}$ should be obtained as follows:

$$\bar{\lambda}_{eff.v} = 0{,}35 + 0{,}7\bar{\lambda}_v \qquad \text{for buckling about v-v axis}$$

$$\bar{\lambda}_{eff.y} = 0{,}50 + 0{,}7\bar{\lambda}_y \qquad \text{for buckling about y-y axis} \qquad \text{(B.1)}$$

$$\bar{\lambda}_{eff.z} = 0{,}50 + 0{,}7\bar{\lambda}_z \qquad \text{for buckling about z-z axis}$$

where $\bar{\lambda}$ is as defined in EN1993-1-1.

(2) When only single bolts are used for end connections of angle web members or when the end connection has poor stiffness, the eccentricity should be taken into account using EN1993-1-1 and the buckling length L_{cr} should be taken as equal to the system length L.

Draft note: Rules to be checked !.

B.2.3 Hollow sections as members

(1) The buckling length L_{cr} of a hollow section chord member should be taken as 0,9L for both in-plane and out-of-plane buckling, where L is the system length for the relevant plane, unless a smaller value is justified by analysis.

(2) The buckling length L_{cr} of an I or H section chord member should be taken as 0,9L for in-plane buckling and 1,0L for out-of-plane buckling, unless a smaller value is justified by analysis.

(3) The buckling length L_{cr} of a hollow section brace member with bolted connections should be taken as 1,0L for both in-plane and out-of-plane buckling.

(4) The buckling length L_{cr} of a hollow section brace member without cropping or flattening, welded around its perimeter to hollow section chords, may generally be taken as 0,75L for both in-plane and out-of-plane buckling. Alternatively its buckling length may be determined by using the expressions given in Table B.1.

(5) If the conditions at each end of brace member differ, the buckling length L_{cr} should be taken as the arithmetic mean of the respective values of the two end conditions.

Table B.1: In-plane and out-plane buckling length factors for hollow section brace members welded to hollow section chords

Chord member	Brace member	L_{cr}/L
CHS	CHS	$\frac{L_{cr}}{L} = 2{,}2\left(\frac{d_1^2}{Ld_0}\right)^{0,25}$ but $L_{cr}/L \geq 0{,}6$ and $L_{cr}/L \leq 0{,}75$
RHS	CHS	$\frac{L_{cr}}{L} = 2{,}35\left(\frac{d_1^2}{Lb_0}\right)^{0,25}$ but $L_{cr}/L \geq 0{,}6$ and $L_{cr}/L \leq 0{,}75$
	RHS	In-plane $\frac{L_{cr}}{L} = 2{,}3\left(\frac{b_1^2}{Lb_0}\right)^{0,25}$ but $L_{cr}/L \geq 0{,}6$ and $L_{cr}/L \leq 0{,}75$ Out-of-plane $\frac{L_{cr}}{L} = 2{,}3\left(\frac{h_1^2}{Lb_0}\right)^{0,25}$ but $L_{cr}/L \geq 0{,}6$ and $L_{cr}/L \leq 0{,}75$

where:
CHS denotes a circular hollow section member,
RHS denotes a rectangular hollow section member,

b_0 is the width of a rectangular hollow section chord member (out-of-plane),
b_1 is the width of a rectangular hollow section brace member (out-of-plane),
d_0 is the diameter of a circular hollow section chord member,
d_1 is the diameter of a circular hollow section brace member,
h_1 is the height of a rectangular hollow section chord member (in-plane),

L is the system length of a brace member.

B.3 Torsional and torsional-flexural buckling

B.3.1 Elastic critical buckling load of compression members

(1) The elastic torsional-flexural buckling force $N_{cr.TF}$ is generally given by the solution of the following cubic equation

$$\left(N_{cr.y} - N\right)\left(N_{cr.z} - N\right)\left(N_{cr.T} - N\right) i_0^2 - z_0^2 N^2 \left(N_{cr.y} - N\right) - y_0^2 N^2 \left(N_{cr.z} - N\right) = 0 \tag{B.2}$$

where $N_{cr.y}$ is the elastic critical force for flexural buckling about the y-y axis

$N_{cr.z}$ is the elastic critical force for flexural buckling about the z-z axis

$$N_{cr.T} = \frac{1}{i_0^2}\left[GI_t + \frac{\pi^2 EI_w}{\ell_T^2}\right]$$

$$i_0^2 = i_y^2 + i_z^2 + y_0^2 + z_0^2$$

G is the shear modulus

I_t is the torsion constant of the gross cross-section

I_w is the warping constant of the gross cross-section

i_y is the radius of gyration of the gross cross-section about the y-y axis

i_z is the radius of gyration of the gross cross-section about the z-z axis

y_0 , z_0 are the shear centre co-ordinates with respect to the centroid of the gross cross-section

ℓ_T is the effective elastic torsional buckling length.

(2) For doubly symmetric cross-sections (e.g. $y_0 = z_0 = 0$)

$$N_{cr.TF} = N_{cr.T} \tag{B.3}$$

provided $N_{cr.T} < N_{cr.y}$ and $N_{cr.z}$.

(3) For cross-sections that are symmetrical about the y-y axis (e.g. $x_0 = 0$)

$$N_{cr.TF} = \frac{N_{cr.y}}{2\beta}\left[1 + \frac{N_{cr.T}}{N_{cr.y}} - \sqrt{\left(1 - \frac{N_{cr.T}}{N_{cr.y}}\right)^2 + 4\left(\frac{y_0}{i_0}\right)^2 \frac{N_{cr.T}}{N_{cr.y}}}\right] \tag{B.4}$$

where $\beta = 1 - \left(\frac{y_0}{i_0}\right)^2$

(4) The buckling length $L_{cr,T}$ for torsional or torsional-flexural buckling shall be determined taking into account the degree of torsional and warping restraint at each end of the system length L_T.

(5) The value of $L_{cr,T}/L_T = 1{,}0$ should be used if not analysed otherwise.

B.3.2 Elastic critical moment for beams

B.3.2.1 Cross-sections symmetrical about the minor axis

(1) In the case of a beam of uniform cross-section that is symmetrical about the minor axis, for bending about the major axis the elastic critical moment for lateral-torsional buckling is given by the general formula:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2}\left\{\sqrt{\left[\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g - C_3 z_j)^2\right]} - [C_2 z_g - C_3 z_j]\right\} \tag{B.5}$$

where $G = \frac{E}{2\ (1+\nu)}$

I_t is the torsion constant

I_w is the warping constant

I_z is the second moment of area about the minor axis

L is the length of the beam between points which have lateral restraint.

C_1, C_2 and C_3 are factors depending on the loading and end restraint conditions

k and k_w are effective length factors

$z_g = z_a - z_s$

$$z_j = z_s - 0{,}5 \int_A \left(y^2 + z^2\right)\frac{z}{I_y} dA$$

z_a is the coordinate of the point of load application

z_s is the coordinate of the shear centre

NOTE See (6) and (7) for sign conventions and (9) for approximations for z_j

(2) The effective length factors k and k_w vary from 0,5 for full restraint to 1,0 for no restraint, with 0,7 for one end fixed and one end free. The normal conditions of restraint at each end are:

$k = k_w = 1,0$

- restrained against lateral movement, free to rotate on plan
- restrained against rotation about the longitudinal axis, free to warp
- restrained against movement in plane of loading, free to rotate in plan

NOTE

- the factor k refers to end rotation on plan. It is analogous to the ratio ℓ/L for a compression member
- the factor k_w refers to end warping

(3) Unless special provision for warping restraint is made, k_w should be taken as 1,0.

(4) Values of C_1, C_2 and C_3 are given in Table B.2 and Table B.3 for various load cases, as indicated by the shape of the bending moment diagram over the length L between lateral restraints. Values are given corresponding to various values of k and in Table B.3 also corresponding to various values of k_w.

(5) For cases with k = 1,0 the value of C_1 for doubly symmetric profiles and any ratio of end moment loading as indicated in Table B.3, is given approximately by:

$$C_1 = 1,88 - 1,40\psi + 0,52\psi^2 \quad \text{but } C_1 \leq 2,70 \qquad \text{(B.6)}$$

(6) The sign convention for determining z_j, see Figure B.3, is:

- z is negative for the compression flange
- z_j is positive when the flange with the larger value of I_z is in compression at the point of largest moment.

Draft note: According to Prof. Ivan Balaz this sentence is not correct and should be deleted – e.g. see fixed beams. z_j is clearly defined by $z_j = z_s - 0,5 \int_A (y^2 + z^2) \frac{z}{I_y} dA$

(7) The sign convention for determining z_g is:

- for gravity loads z_g is negative for loads applied above the shear centre
- in the general case z_g is negative for loads acting towards the shear centre from their point of application.

Draft note: Prof. Ivan Balaz suggests that sign convention should be changed. Reference is also made to DIN 18800 T2 and the format of Eq.(19).

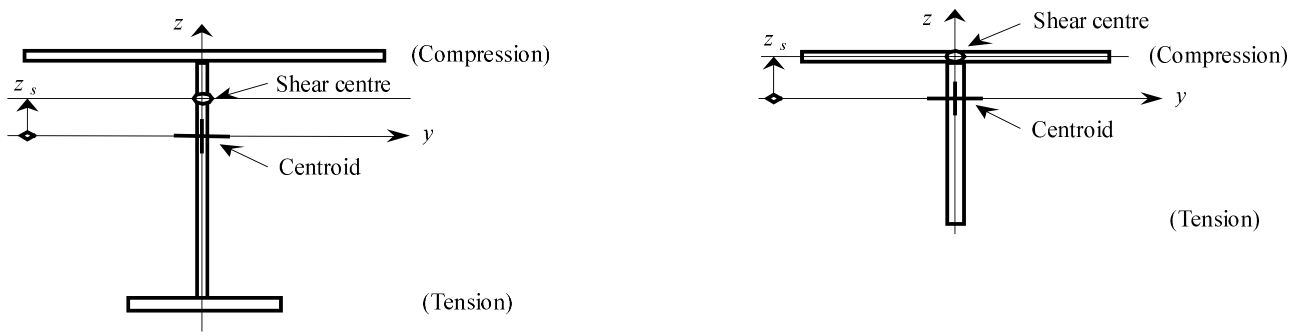


Figure B.3: Sign convention for determining z_j ((Sign convention should be changed))

Draft note: Modifications proposed by Prof. Ivan Balaz

(8) For an I-section with unequal flanges:

$$I_w = \left(1-\psi_f^2\right) I_z \left(\frac{h_s}{2}\right)^2 \tag{B.7}$$

where $\psi_f = \dfrac{I_{fc} - I_{ft}}{I_{fc} + I_{ft}}$

I_{fc} is the second moment of area of the compression flange about the minor axis of the cross-section

I_{ft} is the second moment of area of the tension flange about the minor axis of the cross-section

h_s is the distance between the shear centres of the flanges.

(2) The following approximations for z_j can be used:

$$z_j = 0{,}8\psi_f \frac{h_s}{2} \quad \text{when } \psi_f \geq 0 \tag{B.8}$$

$$z_j = \psi_f \frac{h_s}{2} \quad \text{when } \psi_f < 0 \tag{B.9}$$

for sections with a lipped compression flange:

$$z_j = 0{,}8\psi_f \left(1+\frac{h_L}{h}\right)\frac{h_s}{2} \quad \text{when } \psi_f \geq 0 \tag{B.10}$$

$$z_j = \psi_f \left(1+\frac{h_L}{h}\right)\frac{h_s}{2} \quad \text{when } \psi_f < 0 \tag{B.11}$$

where h_L is the depth of the lip.

B.3.2.2 Bi-symmetric cross sections

(1) The elastic critical moment for lateral-torsional buckling of a beam of uniform symmetrical cross-section with equal flanges, z_j =0:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \sqrt{\left[\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2\right]} - C_2 z_g \right\} \tag{B.12}$$

(2) For end-moment loading $C_2 = 0$ and for transverse loads applied at the shear centre $z_g = 0$. For these cases:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z}} \tag{B.13}$$

(3) For normal conditions of restraint at each end, $k = k_w = 1{,}0$:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}} \tag{B.14}$$

Table B.2: Values of factors C_1, C_2 and C_3 corresponding to values of factor k: End moment loading

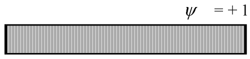
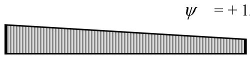
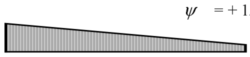
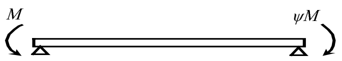
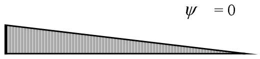
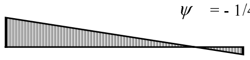
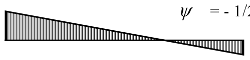
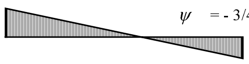
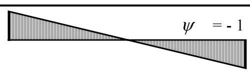
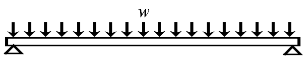
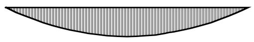
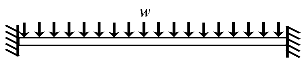
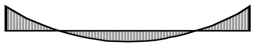
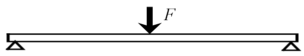
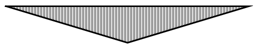
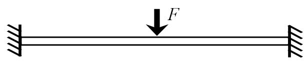
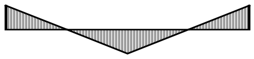
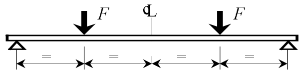
Loading and support conditions	Bending moment diagram	Values of k	Values of factors C_1	C_2	C_3
	$\psi = +1$ 	1,0 0,7 0,5	1,000 1,000 1,000	-	1,000 1,113 1,144
	$\psi = +3/4$ 	1,0 0,7 0,5	1,141 1,270 1,305	-	0,998 1,565 2,283
	$\psi = +1/2$ 	1,0 0,7 0,5	1,323 1,473 1,514	-	0,992 1,556 2,271
	$\psi = +1/4$ 	1,0 0,7 0,5	1,563 1,739 1,788	-	0,977 1,531 2,235
	$\psi = 0$ 	1,0 0,7 0,5	1,879 2,092 2,150	-	0,939 1,473 2,150
	$\psi = -1/4$ 	1,0 0,7 0,5	2,281 2,538 2,609	-	0,855 1,340 1,957
	$\psi = -1/2$ 	1,0 0,7 0,5	2,704 3,009 3,093	-	0,676 1,059 1,546
	$\psi = -3/4$ 	1,0 0,7 0,5	2,927 3,009 3,093	-	0,366 0,575 0,837
	$\psi = -1$ 	1,0 0,7 0,5	2,752 3,063 3,149	-	0,000 0,000 0,000

Table B.3: Values of factors C_1, C_2 and C_3 corresponding to values of factor k: Transverse loading cases

Loading and support conditions	Bending moment diagram	Values of k	Values of factors C_1	C_2	C_3
		1,0 0,5	1,132 0,972	0,459 0,304	0,525 0,980
		1,0 0,5	1,285 0,712	1,562 0,652	0,753 1,070
		1,0 0,5	1,365 1,070	0,553 0,432	1,730 3,050
		1,0 0,5	1,565 0,938	1,267 0,715	2,640 4,800
		1,0 0,5	1,046 1,010	0,430 0,410	1,120 1,890

B.3.3 Minimum restraints along columns and beams

B.3.3.1 Lateral restraints

(1) If trapezoidal sheeting according to EN 1993-1-3 is connected to a beam and the condition expressed by equation (B.15) is met, the beam at the connection may be regarded as being laterally restrained in the plane of the sheeting.

$$S \geq \left(EI_w \frac{\pi^2}{l^2} + GI_t + EI_z \frac{\pi^2}{l^2} 0{,}25h^2 \right) \frac{70}{h^2} \qquad \text{(B.15)}$$

where S is the portion of the shear stiffness provided by the sheeting for the examined beam connected to the sheeting at each rib.

I_w is the warping constant

I_t is the torsion constant

I_z is the second moment of area of the cross section about the minor axis of the cross section

If the sheeting is connected to a beam at every second rib only, S shall be substituted by $0{,}20 \cdot S$.

NOTE: Eq. (B.15) may also be used to determine the lateral stability of beam flanges used in combination with other types of cladding than trapezoidal sheeting, provided that the connections are of suitable design.

Draft note: It has to be checked if the requirements in EN 1993-1-3 are sufficient for the allowance given above.

B.3.3.2 Torsional restraint

Draft note: To be developed to identify product requirements.

B.4 Stable lengths of segment containing plastic hinges for out-of-plane buckling

B.4.1 Uniform members with bi-symmetric I-sections

B.4.1.1 Stable lengths between adjacent lateral restraints

(1) The length L between restraints of a segment of a member containing a plastic hinge can develop full plastic action if L is not greater than L_m , where:

$$L_m = \frac{38 i_z}{\sqrt{\frac{1}{57{,}4}\left(\frac{N_{Ed}}{A}\right) + \frac{756}{C_1^2}\left(\frac{W_{pl}^2}{A I_t}\right)\left(\frac{f_y}{235}\right)^2}} \qquad \text{(B.16)}$$

provided that the member is restrained at the hinge as required by (EN 1993 restraint at a plastic hinge) and that

- either there are lateral restraints at both ends of the segment to the compression flange where one flange is in compression throughout the length of the segment,
- or there are lateral restraints at both ends of the segment and a torsional restraint to the member at a distance that satisfies the requirements for L_s.

(2) For tapered I-sections with uniform flanges, L_m may be calculated using the section properties of the deepest section.

B.4.1.2 Stable length between torsional restraints

(1) For members under constant moment and no axial compression, a segment of a member containing a plastic hinge can develop full plastic action if L is not greater than L_k , provided that

- the member is restrained at the hinge as required by (EN 1993 restraint at a plastic hinge)
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_k = \frac{\left(5{,}4 + \frac{600 f_y}{E}\right)\left(\frac{h}{t_f}\right)}{\sqrt{5{,}4\left(\frac{f_y}{E}\right)\left(\frac{h}{t_f}\right)^2 - 1}} \qquad \text{(B.17)}$$

(2) For members under linear moment gradient and axial compression, a segment of a member containing a plastic hinge can develop full plastic action if L is not greater than L_s , provided that

- the member is restrained at the hinge as required by (EN1993 restraint at a plastic hinge)
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_s = \sqrt{C_m} L_k \left(\frac{M_{pl,Rk}}{M_{pl,N,Rk} + a N_{Ed}}\right) \qquad \text{(B.18)}$$

C_m is the modification factor for linear moment gradient

a is the distance between the centroid of the member with the plastic hinge and the centroid of the restraint members.

(3) For members under non-linear moment gradient and axial compression, a segment of a member containing a plastic hinge can develop full plastic action if L is not greater than L_s , provided that

- the member is restrained at the hinge as required by (EN 1993 restraint at a plastic hinge)
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_s = \sqrt{C_n} L_k \tag{B.19}$$

C_n is the modification factor for non-linear moment gradient, see B 4.3.2.

B.4.2 Haunched or tapered members

(1) For uniform members under linear or non-linear moment gradient and axial compression, a segment of a member containing a plastic hinge can develop full plastic action if L is not greater than L_s , provided that

- the member is restrained at the hinge as required by (EN1993 restraint at a plastic hinge)
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_s = \frac{\sqrt{C_n} L_k}{c} \tag{B.20}$$

L_k is the length derived for a uniform member with a cross-section equal to the shallowest section

c is the value defined in B.4.3.3

B.4.3 Modification factors for moment gradients in members restrained along the tension flange

B.4.3.1 Linear moment gradients

(1) The modification factor Cm may be determined from

$$C_m = \frac{1}{B_0 + B_1\beta_t + B_2\beta_t^{\ 2}} \tag{B.21}$$

in which

$$B_0 = \frac{1 + 10\left(\dfrac{N_{crE}}{N_{crT}}\right)}{1 + 20\left(\dfrac{N_{crE}}{N_{crT}}\right)}$$

$$B_1 = \frac{5\sqrt{\dfrac{N_{crE}}{N_{crT}}}}{\pi + 10\sqrt{\dfrac{N_{crE}}{N_{crT}}}}$$

$$B_2 = \frac{0.5}{1 + \pi\sqrt{\dfrac{N_{crE}}{N_{crT}}}} - \frac{0.5}{1 + 20\left(\dfrac{N_{crE}}{N_{crT}}\right)}$$

$$N_{crE} = \frac{\pi^2 EI_z}{L_t^{\ 2}}$$

L_t is the distance between the torsional restraints

$$N_{crT} = \frac{1}{i_s^{\ 2}}\left(\frac{\pi^2 EI_z a^2}{L_t^{\ 2}} + \frac{\pi^2 EI_w}{L_t^{\ 2}} + GI_t\right)$$

which is the elastic critical buckling force for an I-section between restraints to both flanges at spacing Lt with intermediate lateral restraints to the tension flange.

$$i_s^2 = i_y^2 + i_z^2 + a^2$$

where

a is the distance between the centroid of the member and the centroid of the restraining members, such as rafters.

B.4.3.2 Non linear moment gradients

(1) The modification factor C_n may be determined from

$$C_n = \frac{12}{\left[R_1 + 3R_2 + 4R_3 + 3R_4 + R_5 + 2(R_S - R_E)\right]} \qquad \text{(B.22)}$$

in which R_1 to R_5 are the values of R according to (2) or (3) at the ends, quarter points and mid-length, see Figure B.4, and only positive values of R should be included.

In addition, only positive values of $(R_S – R_E)$ should be included, where

- R_E is the greater of R_1 or R_5
- R_s is the maximum value of R anywhere in the length L_y

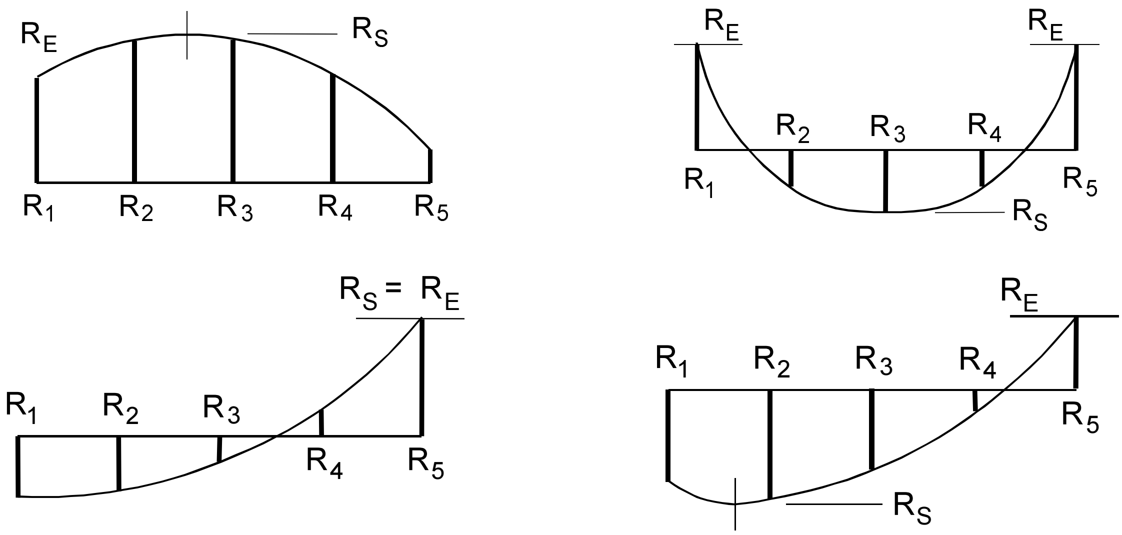


Figure B.4: Moment ratios

(2) When checking the lateral resistance according to EN 1993-1-1. The value of R should be obtained from:

$$R = \frac{M_{y.Sd}}{f_y W_{y.el.c}} \qquad \text{(B.23)}$$

where $W_{y,el,c}$ is the elastic modulus of the section for calculating the compressive stress from major axis moments.

B.4.3.3 Taper factor

(1) For an I-section with $D \geq 1{,}2B$ and $x \geq 20$ the taper factor c should be obtained as follows:

– for tapered members or segments:

$$c = 1 + \frac{3}{x-9}\left(\frac{D_{max}}{D_{min}} - 1\right)^{2/3} \tag{B.24}$$

– for haunched members or segments:

$$c = 1 + \frac{3}{x-9}\left(\frac{D_h}{D_s}\right)^{2/3}\sqrt{\frac{L_h}{L_y}} \tag{B.25}$$

where B is the breadth of the minimum depth cross-section;

D_h is the additional depth of the haunch or taper, see Figure B.5;

D_{max} is the maximum depth of cross-section within the length L_y , see Figure B.5;

D_{min} is the minimum depth of cross-section within the length L_y , see Figure B.5;

D_s is the vertical depth of the un-haunched section, see Figure B.5;

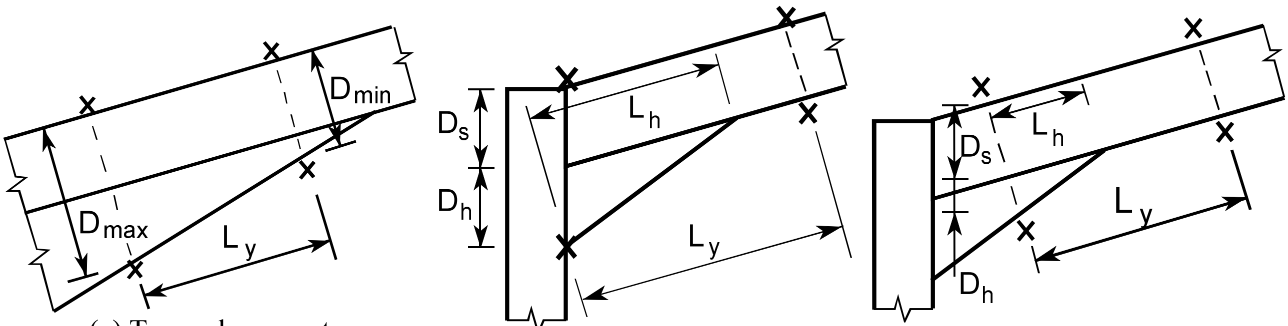
L_h is the length of haunch within the length L_y , see Figure B.5;

L_y is the length between points at which the compression flange is laterally restrained;

x is the torsional index of the minimum depth cross-section, see x.x.x.x.

Draft note: Torsional index to be defined.

x = restraint



(a) Tapered segment (b) Haunched segment (c) Haunched segment

Figure B.5: Dimensions defining taper factor