EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

prEN 1993-3 : 2002

1 May 2002

UDC

Descriptors:

English version

Eurocode 3 : Design of steel structures

Part 3 : Buildings

Calcul des structures en acier

Partie 3 :

Ingènierie du bâtiment

Bemessung und Konstruktion von Stahlbauten

Teil 3 :

Hallen und Geschoßbauten ~~Hochbau~~

Stage 34 draft

Amendments sent prior to the CEN TC 250 / SC 3 meeting in Vienna are highlighted by yellow colour.

Amendments made during the CEN TC 250 / SC 3 meeting in Vienna are highlighted by light blue colour.

CEN

European Committee for Standardisation
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Ref. No. EN 1993-3 : 2002. E

Content

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No existing European Standard is superseded.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.

[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:

- 3.2.3(2)
- 3.2.4(1)
- 6.1(1)
- 6.3.2(2)
- 6.3.2(3)
- 7.2.1(1)
- 7.2.2(1)
- 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 portals for halls and single and multi-storey 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, 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 steel components of composite members covered in EN 1994-1 are also given in this Part 3.

(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 building 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: Joints

[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 joints 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 1990.

1.4 Distinction between principles and application rules

(1) See 1.4 of EN 1990.

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: Will be added later.

1.6 Symbols

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

Draft note: Will be added later.

1.7 Convention for members axis

(1) For member axis see EN 1993-1-1.

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 structure is expected to be used for its intended purpose.

(2) For the specification of the intended design working life of a permanent building see Table 2.1 of EN 1990.

(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 environmental actions and fatigue if relevant or else protected from them.

(2) 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.

(3) 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.

2.1.3.4 Robustness ~~and structural integrity~~

(1) Buildings should be designed for accidental actions ~~design situations as defined in~~ (see EN 1991-1-7).

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.

NOTE For proportional loading see Annex A.

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.~~

3.2.1 Material properties

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

3.2.2 Ductility requirements

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

3.2.3 Fracture toughness

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

(2) For building components under compression a suitable minimum toughness property should be selected.

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

3.2.4 Through thickness properties

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

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

NOTE 2 The National Annex may give the relevant classification according to 3.2 (3) of EN 1993-1-10. The classification in Table 3.1 is recommended:

Table 3.1: Choice of quality class according to EN 10164

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

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 EN Product Standard or European Technical Approval.

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) Corrosion protection does not need to be applied for internal structures, if the internal relative humidity is 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) Portal frames and beam-and-column type plane frames in buildings may be checked for sway mode failure with first order theory if the following criterion is satisfied for each storey:

$$\alpha_{crit} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) \geq 10 \qquad (5.1)$$

where H_{Ed} is the horizontal reaction to the horizontal loads and fictitious horizontal loads at the bottom of the storey

V_{Ed} is the total design vertical load on the structure on the top of the storey

$\delta_{H,E\delta}$ is the horizontal displacement at the top of the storey, relative to the bottom of the storey, when the frame is loaded with horizontal loads (e.g. wind) and fictitious horizontal loads which are applied at each floor level

h is the storey height

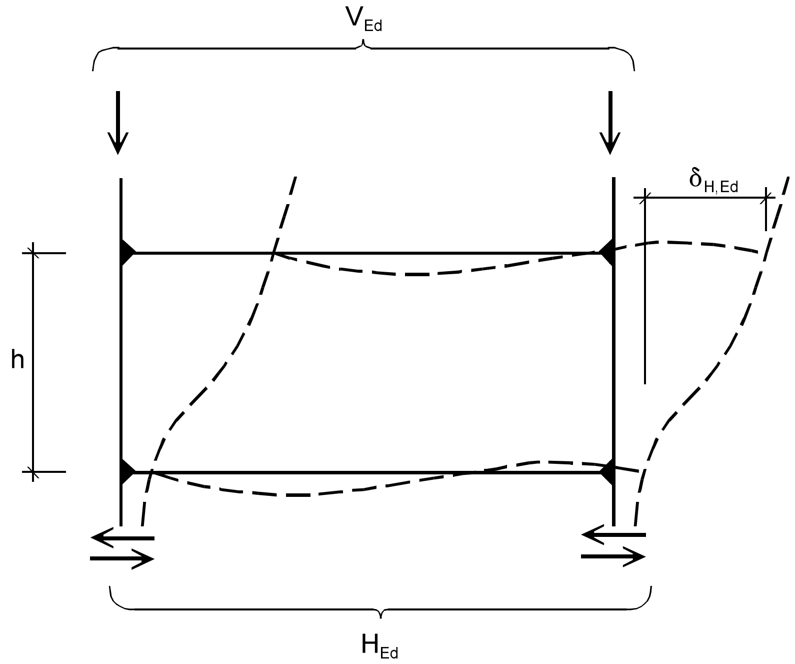


Figure 5.1: Notations for 5.2.1(2)

NOTE 1 For regular multi-storey building frames (see 5.2.2(3)) the check of the base storey is sufficient.

NOTE 2 Non sway mode failure may be relevant for portal frames and the criterion (5.1) not be sufficient where

- the span L exceeds 5 times the mean height h of the columns
- the height of the roof above the straight line between the column tops of each span h_v exceeds 0,25 times the span L
- if the rafter is asymmetric h_v exceeds the limit

$$\left(\frac{h_v}{s_a}\right)^2 + \left(\frac{h_v}{s_b}\right)^2 = 0,5$$

in which s_a and s_b are the horizontal distances from the apex to the columns, see Figure 5.2.

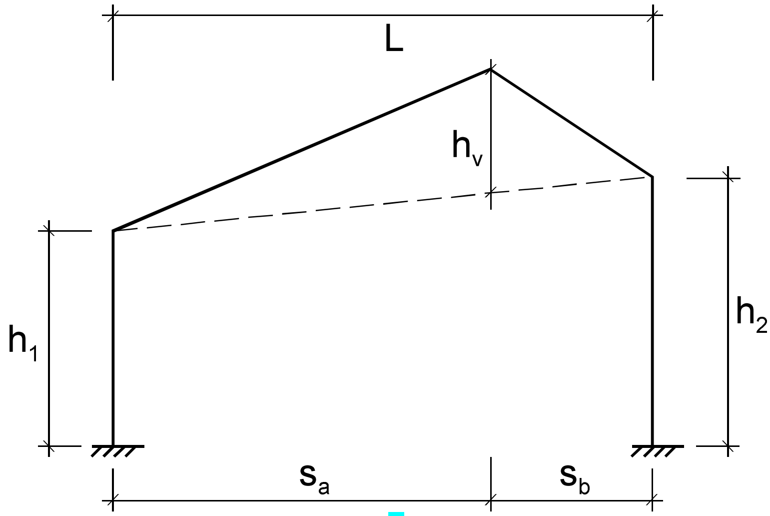


Figure 5.2: Portal frame definitions

5.2.2 Method of analysis

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

(2) For single storey frames second order sway effects may be calculated by means of applying a factor to the horizontal loads H_{Ed} (e.g. wind) and equivalent loads due to imperfections (see 5.2.2 (4)):

$$H = H_0 \frac{1}{1 - \frac{1}{\alpha_{crit}}} \tag{5.2}$$

with $\alpha_{crit} = \left(\frac{H_0}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) \geq 3{,}0$

$$H_0 = H_{Ed} + V_{Ed}\,\phi$$

NOTE For $\alpha_{crit} < 3{,}0$ a true second order analysis applies.

(3) For multi-storey frames second order sway effects may be calculated by means of the method given in (2) provided there is an affinity over all storeys between

- distribution of sum of vertical storey loads and
- distribution of sum of horizontal storey loads and
- distribution of frame stiffness in each storey level due to horizontal loads.

NOTE For the limitation of the method see also 5.2.1 (2).

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 sway imperfections may be disregarded where

$$H_{Ed} \geq 0{,}15\ V_{Ed} \tag{5.3}$$

(3) For the determination of horizontal forces to floor diaphragms the configuration of imperfections as given in Figure 5.3 should be applied, where ϕ refers to sway imperfections of single storeys only, see 5.3.2 of EN 1993-1-1.

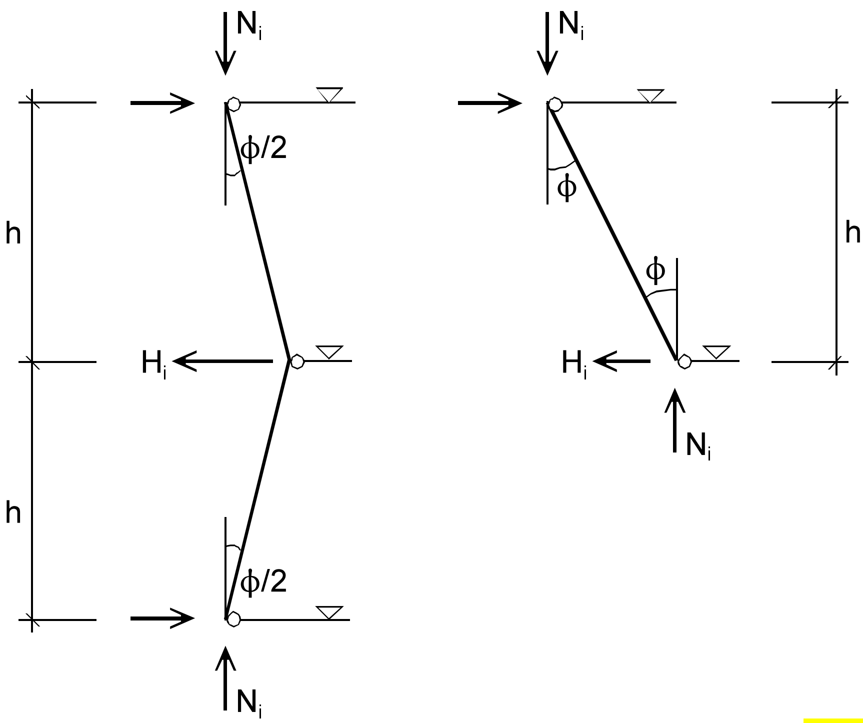


Figure 5.3: Configuration of sway imperfections ϕ for floor diaphragms

(4) In multiple beam-and-column building frames, 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.

5.3.3 Imperfection for analysis of bracing systems

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

5.3.4 Member imperfections in global analysis

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

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 in continuous beams following a first-order analysis, see EN 1993-1-1 5.2.1, the plastic redistribution of moments may be taken into account 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.

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

$\gamma_{M1} = 1{,}00$

$\gamma_{M2} = 1{,}25$

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 elastic 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 with lateral restraint to the compression flange are not susceptible to lateral-torsional buckling if the length L_c between restraints corresponding to the slenderness $\bar{\lambda}_f$ of the equivalent compression flange satisfies:

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

where $M_{y,Ed}$ 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 of 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 of 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

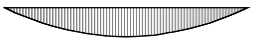
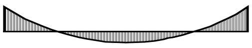
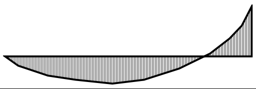
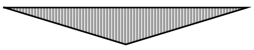
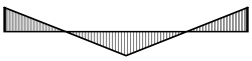
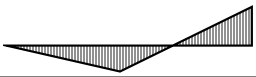
$A_{eff,f}$ is the effective areas of the flange

$A_{eff,w,c}$ 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 $\bar{\lambda}_{c0} = \bar{\lambda}_{LT,0} + 0{,}1$ is recommended, see EN 1993-1-1, 6.3.2.3.

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

shape of moment distribution	class 1, 2, 3 sections k_c	class 4 sections k_c
 $\psi = 1$	1,0	1,0
 $-1 \le \psi \le 1$	$\frac{1}{1{,}33 - 0{,}33\psi}$	$\frac{1}{1{,}33 - 0{,}33\psi}$
	0,94	1,0
	0,90	1,0
	0,91	1,0
	0,86	1,0
	0,77	1,0
	0,82	1,0
~~**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:

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

where χ is the reduction factor of the equivalent compression flange determined with $\bar{\lambda}_f$, see (2)

$k_{f\ell}$ is the modification factor accounting for the conservatism of the equivalent compression flange method

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

curve c for all other sections

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 also Annex B.3.

6.3.3 Lateral torsional buckling of frames

6.3.3.1 General method

(1) See 6.3.3 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, see 5.3.4 of EN 1993-1-1

b) verification of stable length of segment between such restraints and other lateral 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 h/2 along the length of the member, where h 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. For non uniform segments see 6.3.3 of EN 1993-1-1.

(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 h/2 along the length of the tapered segment, not 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 axial load in the member

b) the length L_y to the adjacent lateral restraint to the compression flange does not exceed 85% of the limiting length

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 limits for vertical deflections should be agreed.

NOTE The National Annex may specify the limits.

7.2.2 Recommendations for horizontal deflections

(1) With reference to EN 1990 – Annex A 1.4.3 – Figure A.11 limits for horizontal deflections should be agreed.

NOTE The National Annex may specify the limits.

7.2.3 Dynamic effects

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

NOTE 1 The National Annex may specify limits for vibration of floors.

8 ~~Fasteners, welds, connections and~~Design of joints

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

Annex A [informative] – Additional design provisions

A.1 Structural analysis taking account of material non-linearities

(1) In case of material non-linearities the action effects in a structure may be determined by incremental approach to the design loads to be considered for the relevant design situation.

(2) In this incremental approach each permanent and variable action should be increased proportionally.

A.2 Simplified provisions for the design of continuous floor beams

(1) For continuous beams with slabs in buildings without cantilevers on which uniformly distributed loads are dominant, it will be sufficient to consider only the following load arrangements:

a) alternative spans carrying the design permanent and variable load ($\gamma_G\ G_k + \gamma_Q\ Q_k$), other spans carrying only the design permanent load $\gamma_G\ G_k$

b) any two adjacent spans carrying the design permanent and variable loads ($\gamma_G\ G_k + \gamma_Q\ Q_k$), all other spans carrying only the design permanent load $\gamma_G\ G_k$

NOTE 1 a) applies to sagging moments, b) to hogging moments.

NOTE 2 This annex is intended to be transferred to EN 1990 in a later stage.

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.

(3) If used for single portal frames in which the loading is carried by bending of the rafters, these formulae do not consider the destabilising effect of the axial compression in the rafters. Where the span/column height is greater than 5, this causes a significant reduction in capacity of the frame due to second order effects.

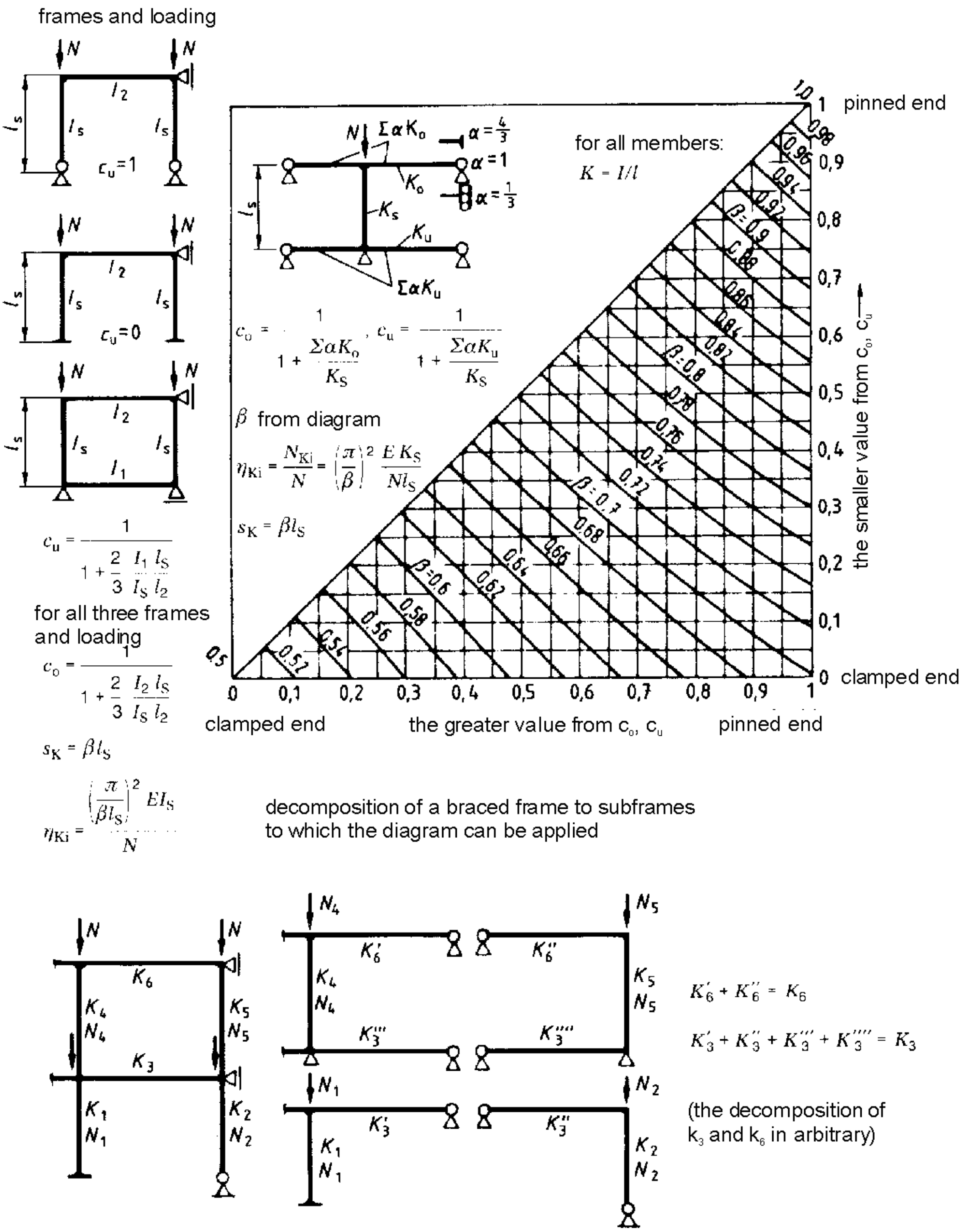


Figure B.1: Buckling length ratio β = L_{cr} / L for a column in a non-sway mode

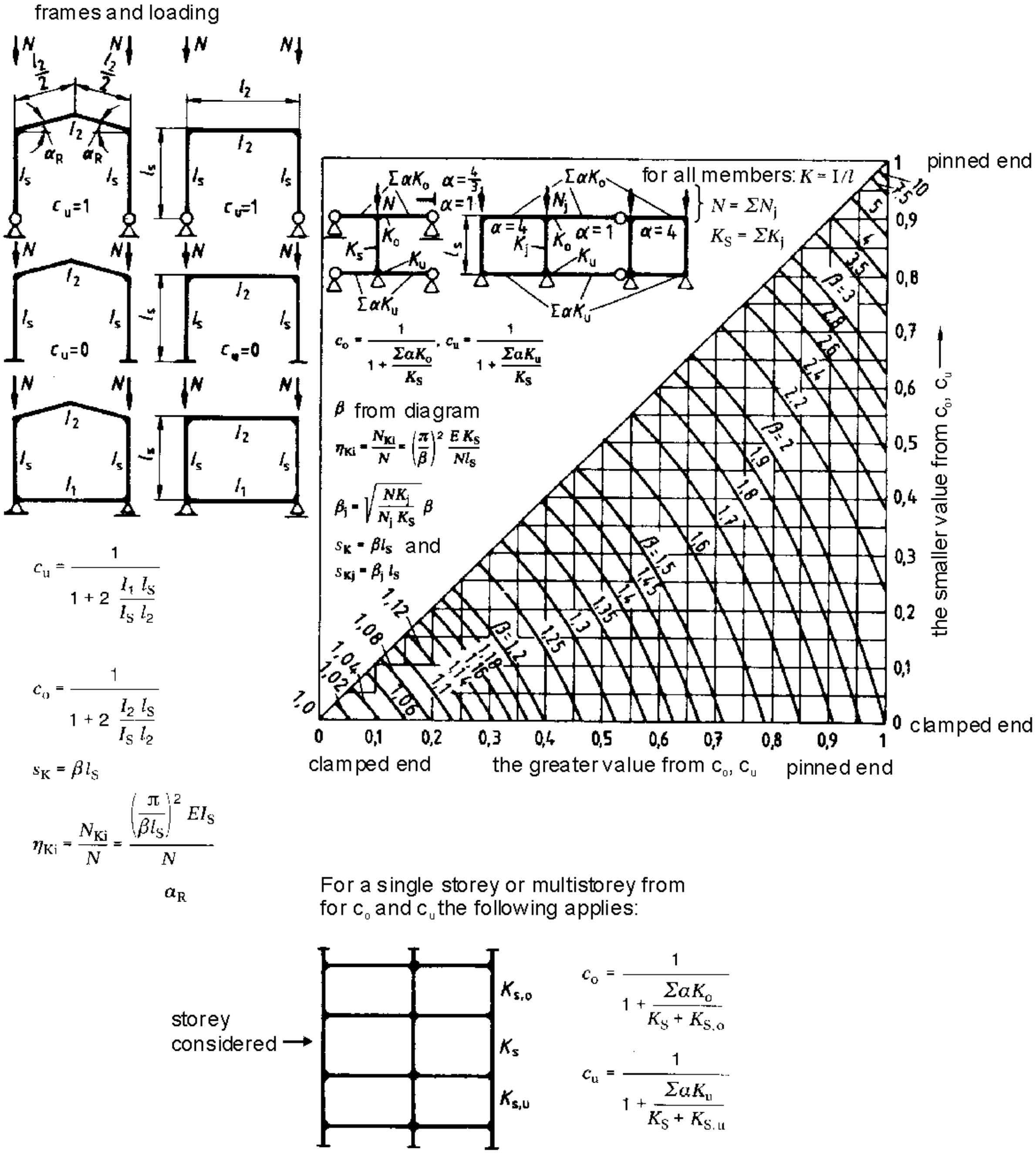


Figure B.2: Buckling length ratio $\beta = L_{cr} / L$ for a column in a sway mode

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} may be taken as equal to the system length L, unless a smaller value is justified by analysis.

NOTE In hollow section latticed girders web members are braced members.

(2) Web members may be designed for 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 $\bar{\lambda}_{eff}$ may be obtained as follows:

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

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

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

where $\bar{\lambda}$ is as defined in EN 1993-1-1.

(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.

B.2.3 Hollow sections as members

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

(2) The buckling length L_{cr} of an I or H section chord member may 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 may 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 ~~0,75~~0,90L for both in-plane and out-of-plane buckling, unless smaller values may be justified by tests or by calculations. ~~For brace to chords diameter or width ratios less than 0,6 the buckling length may alternatively be determined by using the expressions given in Figure B.1.~~

B.3 Minimum restraints along columns and beams

B.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.2) is met, the beam at the connection may be regarded as being laterally restrained in the plane of the sheeting.

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

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 should be substituted by $0{,}20 \cdot S$.

NOTE Eq. (B.2) 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.

B.3.2 Torsional restraint

(1) A frame member may be considered as sufficiently restraint from rotational deformations if

$$C_{\vartheta,k} > \frac{M_{pl,k}^2}{EI_z} K_\vartheta K_\upsilon \tag{B.3}$$

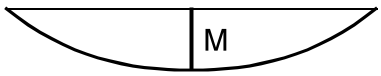
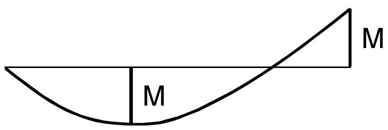
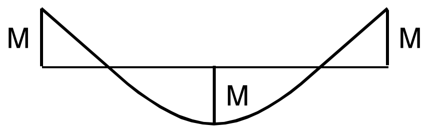
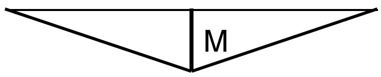
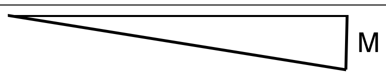
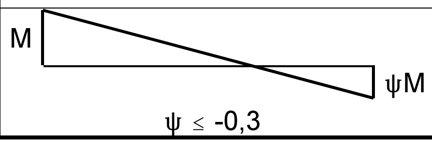
where $C_{\vartheta,k}$ = rotational stiffness of the roof structure

K_υ = 0,35 for elastic analysis ~~1 applicable for elastic analysis methods~~

K_υ = 1,00 for plastic analysis

K_ϑ = factor for considering the moment distribution see Table B.1 and the type of restraint.

Table B.1: Factor K_ϑ for considering the moment distribution and the type of restraint

Case	Moment distribution	without translational restraint	with translational restraint
1		4,0	0
2a		3,5	0,12
2b			0,23
3		2,8	0
4		1,6	1,0
5		1,0	0,7

(2) The rotational stiffness of the roof structure may be calculated from

$$\frac{1}{C_{\vartheta,k}} = \frac{1}{C_{\vartheta R,k}} + \frac{1}{C_{\vartheta C,k}} + \frac{1}{C_{\vartheta D,k}} \tag{B.4}$$

where $C_{\vartheta R,k}$ = rotational stiffness of the roof structure assuming a stiff connection to the member

$C_{\vartheta C,k}$ = rotational stiffness of connection between the member and the roof structure

$C_{\vartheta D,k}$ = rotational stiffness due to distorsional deformations of the member

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

B.4.1 Uniform members made of rolled sections ~~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:

$$L_m = \frac{38 i_z}{\sqrt{\frac{1}{57,4}\left(\frac{N_{Ed}}{A}\right) + \frac{756}{C_1^2}\left(\frac{W_{pl}^2}{A I_t}\right)\left(\frac{f_y}{235}\right)^2}} \qquad \text{(B.5)}$$

provided that the member is restrained at the hinge as required by EN 1993-1-1 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 1993-1-1
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_k = \frac{\left(5,4 + \frac{600 f_y}{E}\right)\left(\frac{h}{t_f}\right) i_z}{\sqrt{5,4\left(\frac{f_y}{E}\right)\left(\frac{h}{t_f}\right)^2 - 1}} \qquad \text{(B.6)}$$

(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 EN1993-1-1
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_s = \sqrt{C_m} L_k \left(\frac{M_{pl,Rk}}{M_{pl,N,Rk} + aN_{Ed}} \right) \tag{B.7}$$

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 1993-1-1
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_s = \sqrt{C_n} L_k \tag{B.8}$$

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 EN1993-1-1
- there are one or more intermediate restraints between the torsional restraints at a spacing that satisfies the requirements for L_m ,

where

$$L_s = \frac{\sqrt{C_n} L_k}{c} \tag{B.9}$$

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

c is the taper factor 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

$$C_m = \frac{1}{B_0 + B_1\beta_t + B_2\beta_t^{\,2}} \tag{B.10}$$

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

$$C_n = \frac{12}{\left[R_1 + 3R_2 + 4R_3 + 3R_4 + R_5 + 2\left(R_S - R_E\right)\right]} \qquad \text{(B.11)}$$

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

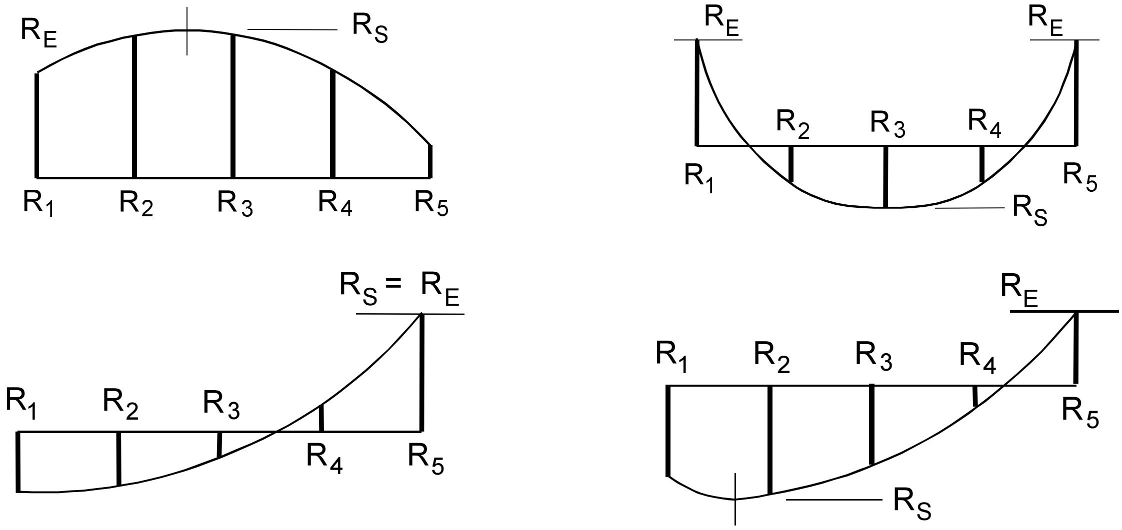


Figure B.3: Moment ratios

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

$$R = \frac{M_{y,Ed} + a\ N_{Ed}}{f_y W_{y,c}} \qquad \text{(B.12)}$$

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

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

B.4.3.3 Taper factor

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

– for tapered members or segments:

$$c = 1 + \frac{3}{\frac{h}{t_f} - 9}\left(\frac{D_{max}}{D_{min}} - 1\right)^{2/3} \tag{B.13}$$

– for haunched members or segments:

$$c = 1 + \frac{3}{\frac{h}{t_f} - 9}\left(\frac{D_h}{D_s}\right)^{2/3}\sqrt{\frac{L_h}{L_y}} \tag{B.14}$$

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.4;

D_{max} is the maximum depth of cross-section within the length L_y, see Figure B.4;

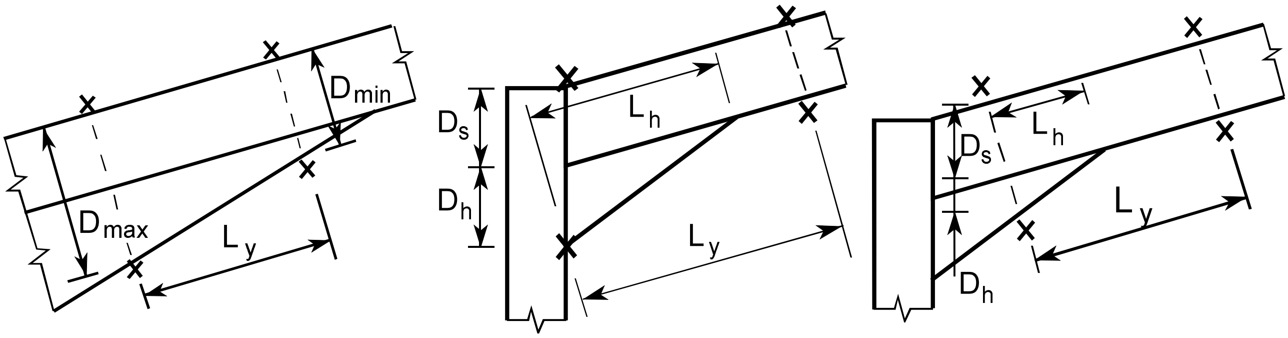
D_{min} is the minimum depth of cross-section within the length L_y, see Figure B.4;

D_s is the vertical depth of the un-haunched section, see Figure B.4;

L_h is the length of haunch within the length L_y, see Figure B.4;

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

x = restraint



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

Figure B.4: Dimensions defining taper factor