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Eurocode 3 - Design of steel structures - Part 1-4: Stainless steel structures

Eurocode 3 - Calcul des structures en acier - Partie 1-4: Structures en aciers inoxydables

Eurocode 3 - Bemessung und Konstruktion von Stahlbauten - Teil 1-4: Tragwerke aus nichtrostenden Stählen

This draft European Standard is submitted to CEN members for enquiry. It has been drawn up by the Technical Committee CEN/TC 250.

If this draft becomes a European Standard, CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration.

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EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

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European foreword

This document (prEN 1993-1-4:2023) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1993-1-4:2006 and its amendments.

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

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

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

0 Introduction

0.1 Introduction to the Eurocodes

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

— EN 1990 Eurocode: Basis of structural and geotechnical design

— EN 1991 Eurocode 1: Actions on structures

— EN 1992 Eurocode 2: Design of concrete structures

— EN 1993 Eurocode 3: Design of steel structures

— EN 1994 Eurocode 4: Design of composite steel and concrete structures

— EN 1995 Eurocode 5: Design of timber structures

— EN 1996 Eurocode 6: Design of masonry structures

— EN 1997 Eurocode 7: Geotechnical design

— EN 1998 Eurocode 8: Design of structures for earthquake resistance

— EN 1999 Eurocode 9: Design of aluminium structures

— New parts are under development, e.g. Eurocode for design of structural glass

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

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

0.2 Introduction to EN 1993 (all parts)

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

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

EN 1993 is subdivided in various parts:

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

EN 1993-2, *Design of Steel Structures — Part 2: Steel bridges*;

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

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

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

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

EN 1993-7, *Design of steel structures — Part 7: Design of sandwich panels*.

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

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

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

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

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

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

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

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

EN 1993-1-7, *Design of Steel Structures — Part 1-7: Strength and stability of planar plated structures transversely loaded*;

EN 1993-1-8, *Design of Steel Structures — Part 1-8: Design of joints*;

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

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

EN 1993-1-11, *Design of Steel Structures — Part 1-11: Design of structures with tension components*;

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

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

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

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

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

0.3 Introduction to prEN 1993-1-4

prEN 1993-1-4 gives supplementary rules for the structural design of steel structures that extend and modify the application of EN 1993-1-1, EN 1993-1-3, EN 1993-1-5 and EN 1993-1-8 to stainless steels. The focus is on design methods and design rules for members and skeletal structures regarding resistance and stability.

0.4 Verbal forms used in the Eurocodes

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

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

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

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

0.5 National Annex for prEN 1993-1-4

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

The national standard implementing prEN 1993-1-4 can have a National Annex containing all national choices to be used for the design of steel structures to be constructed in the relevant country.

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

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

National choice is allowed in prEN 1993-1-4 through the following clauses:

5.1.1(1)	5.2.2(1)	7.2.1(1)	7.4.3.5(3)
8.1(1)	A.2(8)	A.3, Table A.4	

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

1 Scope

1.1 Scope of prEN 1993-1-4

This document provides supplementary rules for the structural design of steel structures that extend and modify the application of EN 1993-1-1, EN 1993-1-3, EN 1993-1-5 and EN 1993-1-8 to austenitic, duplex (austenitic-ferritic) and ferritic stainless steels.

NOTE 1 Austenitic-ferritic stainless steels are commonly known as duplex stainless steels. The term duplex stainless steel is used in this document.

NOTE 2 Information on the durability of stainless steels is given in Annex A.

NOTE 3 The execution of stainless steel structures is covered in EN 1090-2 and EN 1090-4.

1.2 Assumptions

Unless specifically stated, EN 1990, EN 1991 (all parts), EN 1993-1-1, EN 1993-1-3, EN 1993-1-5 and EN 1993-1-8 apply.

The design methods given in prEN 1993-1-4 are applicable if

— the execution quality is as specified in EN 1090-2 and EN 1090-4, and

— the construction materials and products used are as specified in EN 1993-1-1, EN 1993-1-3, EN 1993-1-5 and EN 1993-1-8, or in the relevant material and product specifications.

2 Normative references

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

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

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

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

EN 1990, *Basis of structural and geotechnical design*

EN 1991 (all parts), *Actions on structures*

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

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

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

EN 1993-1-8, *Design of Steel Structures — Part 1-9: Fatigue*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, EN 1993-1-1, EN 1993-1-3, EN 1993-1-5 and EN 1993-1-8 apply.

3.2 Symbols and abbreviations

3.2.1 General

For the purposes of this document, the symbols given in EN 1990, EN 1993-1-1, EN 1993-1-3, EN 1993-1-5 and EN 1993-1-8 and the following apply.

3.2.2 Latin upper-case symbols

A_{c}	total corner cross-sectional area of the section
CRF	corrosion resistance factor
CSM	continuous strength method
C_1, C_2, C_3	material coefficients used to define the CSM material model
E_{s}	secant modulus
E_{sh}	strain hardening modulus
$E_{\mathrm{s,ser}}$	secant modulus of elasticity used for serviceability limit state calculations
$E_{\mathrm{s,1}}$	secant modulus corresponding to the stress in the tension flange
$E_{\mathrm{s,2}}$	secant modulus corresponding to the stress in the compression flange
$F_{\mathrm{p,S}}$	preloading force for bolts
F_{Rd}	design value of the resistance of the structure calculated from F_{Rk}
F_{Rk}	characteristic value of the resistance of the structure, taken at the peak load factor attained during the plastic zone analysis or the point at which the CSM strain limit is reached, whichever is the lesser
F_1	risk of exposure to chlorides from salt water or deicing salts
F_2	risk of exposure to sulfur dioxide
F_3	cleaning regime or exposure to washing by rain
K	initial lateral stiffness of the structure
K_{s}	secant lateral stiffness of the structure at the design load level
KV	impact energy in Joule [J] from a Charpy V notch specimen
$L_{\mathrm{b,cs}}$	local buckling half-wavelength
M	distance from the sea

$M_{\mathrm{csm,Rd}}$	design value of the CSM resistance to bending moment about one principal axis of a cross-section
$M_{\mathrm{csm,y,Rd}}$	design value of the CSM bending moment resistance about major (y-y) axis
$M_{\mathrm{csm,z,Rd}}$	design value of the CSM bending moment resistance about minor (z-z) axis
$M_{\mathrm{N,csm,y,Rd}}$	design value of the reduced CSM bending moment resistance about major (y-y) axis making allowance for the presence of axial force
$M_{\mathrm{N,csm,z,Rd}}$	design value of the reduced CSM bending moment resistance about minor (z-z) axis making allowance for the presence of axial force
$N_{\mathrm{csm,Rd}}$	design value of the CSM resistance to axial force of the cross-section for uniform compression
$N_{\mathrm{csm,t,Rd}}$	design value of the CSM resistance to tension axial force
$R_{\mathrm{p0,2}}$	stress at which the plastic extension is 0,2 % (i.e. 0,2 % proof strength), taken from the product standard
S	distance from roads with deicing salts
Y	factor that approximates loss of stiffness due to second order effects

3.2.3 Latin lower-case symbols

f_{ua}	average ultimate tensile strength, accounting for work hardening due to cold-forming
f_{ub}	nominal ultimate tensile strength of stainless steel bolts
f_{yb}	nominal yield strength of stainless steel bolts
f_{yCHS}	enhanced yield strength of a cold rolled circular hollow section
f_{yc}	enhanced yield strength of the corner region
f_{yf}	enhanced yield strength of the flat portions of cold-rolled rectangular hollow sections
k_{s}	coefficient for calculating the slip resistance of preloaded bolts
n	strain hardening exponent
n_{c}	number of 90° corners in the section
n_{csm}	ratio of the design compression force N_{Ed} to the CSM compression resistance $N_{\mathrm{csm,Rd}}$
n_{p}	is the exponent used in the calculation of the enhanced yield strength

3.2.4 Greek upper-case symbols

Ω	a project specific parameter that defines the permissible level of plastic deformation

3.2.5 Greek lower-case symbols

α	CSM bending parameter
$\alpha_{\mathrm{cr,sw,mod}}$	modified factor by which the design load would have to be increased to cause elastic instability in a global (sway) mode to account for the influence of plasticity on the sway stiffness of frame
α_{csm}	CSM interaction coefficient for biaxial bending
β_{csm}	CSM interaction coefficient for biaxial bending
β_{w}	correlation factor for fillet welded connections
$\varepsilon_{\mathrm{CHS}}$	strain induced in a circular hollow section during section forming
ε_{c}	strain induced in the corner region during section forming
$\varepsilon_{\mathrm{csm}}$	limiting compressive strain from the continuous strength method
$\varepsilon_{\mathrm{csm,max}}$	maximum design CSM strain
$\varepsilon_{\mathrm{csm,t}}$	maximum attainable CSM tensile strain
$\varepsilon_{\mathrm{Ed}}$	design strain
ε_{f}	strain induced in the flat faces of rectangular hollow sections during section forming
$\varepsilon_{\mathrm{p0,2}}$	is the total strain corresponding to the 0,2 % proof strength
ε_{u}	ultimate strain, corresponding to the ultimate tensile strength f_{u}
ε_{y}	elastic strain at the yield strength
$\bar{\lambda}_0$	limiting relative slenderness
$\bar{\lambda}_{\mathrm{m}}$	relative slenderness of the member determined at the critical cross-section m
$\bar{\lambda}_{\mathrm{c,cs}}$	cross-section slenderness for circular hollow sections
$\bar{\lambda}_{\mathrm{p,cs}}$	cross-section slenderness for sections comprising flat plates
μ	slip factor
ρ_{csm}	reduction factor to account for the interaction between bending and shear
$\sigma_{\mathrm{cr,c}}$	elastic buckling stress of the full cross-section of the circular hollow section
$\sigma_{\mathrm{cr,cs}}$	elastic local buckling stress of the full cross-section
$\sigma_{i,\mathrm{Ed,ser}}$	serviceability design stress

4 Basis of design

4.1 General rules

4.1.1 Basic requirements

(1) The design of stainless steel structures shall be in accordance with the general rules given in EN 1990 and EN 1991 (all parts) and the specific design provisions for steel structures given in EN 1993-1-1, EN 1993-1-3, EN 1993-1-5 and EN 1993-1-8.

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

4.2 Design assisted by testing

(1) Prototypes for testing should be produced in a similar manner to the components of the final product, such that they reflect the same levels of work hardening.

(2) Due to the anisotropy of cold worked stainless steels, test specimens should be prepared from the plate or sheet in the same orientation (i.e. transverse or parallel to the rolling direction) as intended for the final structure. If the final orientation is unknown or cannot be guaranteed, tests should be conducted for both orientations and the less favourable result should be adopted.

5 Materials

5.1 Structural stainless steels

5.1.1 General

(1) For the design of austenitic, duplex and ferritic stainless steel structures according to this document, the material should conform with one of the grades in Table A.3, and with one of the following product standards: EN 10088, EN 10028-7, 10296-2 and 10297-2.

NOTE 1 The most common stainless steel grades from Table A.3 are listed in Table 5.1 in accordance with their Corrosion Resistance Class and Strength Class.

NOTE 2 Other stainless steel grades and products can be defined in the National Annex.

(2) If other stainless steel grades than those mentioned in (1) are used, their mechanical properties shall conform to the conditions given in 5.1.2.1(4), 5.1.3 and 5.1.4 when tested in accordance with the relevant EN, ISO or EN ISO testing standards. If relevant, specialist advice should be sought regarding the durability, fabrication, weldability, fatigue resistance and high temperature performance of these grades.

Table 5.1 — Strength and corrosion classes for common stainless steels

Corrosion Resistance Class (see Annex A)	Strength Class			
	SC210		SC450	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
	210	500	450	650
I	1.4003[a] (F)			
II	1.4301 (A) 1.4307 (A)		1.4482 (D)	
III	1.4401 (A) 1.4404 (A) 1.4435 (A) 1.4571 (A)		1.4162 (D) 1.4362[b] (D) 1.4062 (D)	
IV			1.4462 (D) 1.4662 (D)	
V			1.4410 (D) 1.4501 (D)	

NOTE 1 F = ferritic, A = austenitic and D = duplex stainless steels.

NOTE 2 The most common austenitic stainless steels are 1.4301/1.4307 and 1.4401/1.4404.

NOTE 3 The strengths apply to sheet, plate, and strip, and products fabricated from sheet, plate and strip. The strengths also apply to cold-formed hollow sections where the strength enhancement arising from fabrication is not taken into account.

NOTE 4 For rods, bars, hot rolled open sections and seamless tubes, f_y = 180 N/mm^2 for the grades in Strength Class SC210, and EN 10088 gives values for f_y for grades in Strength Class SC450.

NOTE 5 Table A.3 categorises a wider selection of stainless steels into Corrosion Resistance Classes.

a f_y = 250-280 N/mm^2 for 1.4003, depending on the product form, and f_u = 450 N/mm^2. For rod, only f_u values apply.

b f_y = 400 N/mm^2

5.1.2 Material properties

5.1.2.1 General

(1) The nominal values of the yield strength f_y and the ultimate tensile strength f_u for stainless steel should be obtained:

a) either by using the Strength Classes given in Table 5.1;

b) or by adopting the values $f_y = R_{p0,2}$ and $f_u = R_m$ (as lower bound of the given range) directly from the product standard.

NOTE The product standard can give higher strength values than Table 5.1, depending on the product form.

The strength of grades not listed in Table 5.1 should be taken from the product standard.

(2) Higher strength values may be derived from cold working of the sheet material, as given in 5.1.2.2 and Table 5.2 or during fabrication of cold-formed sections, as given in 5.1.2.3.

(3) The strength enhancement from cold working should not be used for members which are to be welded or subject to heat treatment unless the localised reduction in strength due to annealing is determined from tests on the full-cross-section.

(4) The ductility requirements in EN 1993-1-1 shall also apply to stainless steels. Steels conforming to one of the steel grades listed in Table 5.1 may be assumed to satisfy these requirements. The steels listed in Table 5.2 should have declared properties that meet the ductility requirements given in EN 1993-1-1.

(5) Where cold working increases the member resistance (see 5.1.2.3), the design of joints should be consistent with this increased member resistance, especially where capacity design is required.

5.1.2.2 Material in the cold worked condition

(1) This document covers the design of austenitic stainless steel material in the cold worked conditions CP350 and CP500. The nominal values of the yield strength and the ultimate tensile strength are given in Table 5.2.

(2)According to EN 10088-4, the CP classification only defines the required 0,2% proof strength, f_y. Therefore, the steel should have declared properties that meet the conservative tabulated values for ultimate tensile strength, f_u, unless type testing demonstrates the acceptability of higher values.

Table 5.2 — Nominal values of the yield strength f_y and the ultimate tensile strength f_u for austenitic stainless steels in the cold worked condition

Cold Worked Condition	f_y (N/mm²) [a]			f_u (N/mm²)
	Grade	Tension	Compression	
CP350	1.4301, 1.4541, 1.4401, 1.4571	350	315	600
CP500	1.4301, 1.4541, 1.4401, 1.4571	470	350	650
	1.4318	430		

[a] EN 10088 gives minimum specified 0,2% proof strengths in tension in the transverse direction of 350 N/mm² and 500 N/mm² for CP350 and CP500, respectively. The reduced values for f_y given in this table account for the anisotropic and asymmetric behaviour of cold worked material. No reduction is required in f_u.

5.1.2.3 Material properties of cold-formed sections and sheeting

(1) An average yield strength f_{ya} and average ultimate tensile strength f_{ua} may be used in place of f_y and f_u to account for the strength enhancement arising during the fabrication of cold-formed structural sections, except for material in the cold worked conditions CP350 and CP500 (see Table 5.2).

(2) f_{ya} is given by:

$$f_{ya} = \frac{f_{yc}\, A_c + f_{yf}(A - A_c)}{A} \quad \text{For press braked sections or cold rolled rectangular hollow sections} \tag{5.1}$$

$$f_{ya} = f_{yCHS} \quad \text{For cold rolled circular hollow sections} \tag{5.2}$$

where

A is the gross cross-sectional area of the section

A_c is the total corner cross-sectional area of the section, given as:

$$A_{\mathrm{c}} = \left(n_{\mathrm{c}}\,\pi\frac{t}{4}\right)(2r+t) \qquad \text{For press-braked sections} \tag{5.3}$$

$$A_{\mathrm{c}} = \left(n_{\mathrm{c}}\,\pi\frac{t}{4}\right)(2r+t) \; + \; 4n_{\mathrm{c}}t^{2} \qquad \text{For cold rolled rectangular hollow sections} \tag{5.4}$$

in which

n_{c} is the number of 90° corners in the section;

r is the internal bend radius, which may be taken as $2t$ if not known;

f_{yc} is the enhanced yield strength of the corner region, obtained from:

$$f_{\mathrm{yc}} = 0{,}85f_{\mathrm{y}}\left(\frac{\varepsilon_{\mathrm{c}}}{\varepsilon_{\mathrm{p0,2}}}+1\right)^{n_{\mathrm{p}}} \qquad \text{but} \quad f_{\mathrm{y}} \le f_{\mathrm{yc}} \le f_{\mathrm{u}} \tag{5.5}$$

f_{yf} is the enhanced yield strength of the flat portions of cold rolled rectangular hollow sections, obtained according to Formula (5.6). For press-braked sections f_{yf} should be taken as f_{y}

$$f_{\mathrm{yf}} = 0{,}85f_{\mathrm{y}}\left(\frac{\varepsilon_{\mathrm{f}}}{\varepsilon_{\mathrm{p0,2}}}+1\right)^{n_{\mathrm{p}}} \qquad \text{but} \quad f_{\mathrm{y}} \le f_{\mathrm{yf}} \le f_{\mathrm{u}} \tag{5.6}$$

f_{y} is the yield strength of the basic material (see 5.1.2.1(1));

f_{yCHS} is the enhanced yield strength of a cold rolled circular hollow section, obtained from:

$$f_{\mathrm{yCHS}} = 0{,}85f_{\mathrm{y}}\left(\frac{\varepsilon_{\mathrm{CHS}}}{\varepsilon_{\mathrm{p0,2}}}+1\right)^{n_{\mathrm{p}}} \qquad \text{but} \quad f_{\mathrm{y}} \le f_{\mathrm{yCHS}} \le f_{\mathrm{u}} \tag{5.7}$$

in which

$$n_{\mathrm{p}} = \frac{\ln(f_{\mathrm{y}}/f_{\mathrm{u}})}{\ln(\varepsilon_{\mathrm{p0,2}}/\varepsilon_{\mathrm{u}})} \tag{5.8}$$

$$\varepsilon_{p0,2} = 0{,}002 + \frac{f_{\mathrm{y}}}{E} \tag{5.9}$$

E is modulus of elasticity, see 5.1.5;

f_{u} is the ultimate tensile strength of the basic material (see 5.1.2.1(1));

ε_{c} is the strain induced in the corner region during section forming, given by:

$$\varepsilon_{\mathrm{c}} = \frac{t}{2(2r+t)} \tag{5.10}$$

ε_{f} is the strain induced in the flat faces of rectangular hollow sections during section forming, given by Formula (5.11), where all dimensions shall be in millimetres:

$$\varepsilon_{\mathrm{f}} = \left[\frac{t}{900}\right] + \left[\frac{\pi t}{2(b + h - 2t)}\right] \tag{5.11}$$

$\varepsilon_{\mathrm{CHS}}$ is the strain induced in a circular hollow section during section forming, given by:

$$\varepsilon_{\mathrm{CHS}} = \frac{t}{2(d - t)} \tag{5.12}$$

ε_{u} is the ultimate strain, corresponding to the ultimate tensile strength f_{u}, which may be approximated as follows:

a) for austenitic and duplex stainless steels

$$\varepsilon_{u} = 1 - \frac{f_{\mathrm{y}}}{f_{\mathrm{u}}} \qquad \text{but} \quad \varepsilon_{\mathrm{u}} \leq \varepsilon_{\mathrm{f}} \tag{5.13}$$

b) for ferritic stainless steels

$$\varepsilon_{u} = 0{,}6\left[1 - \frac{f_{\mathrm{y}}}{f_{\mathrm{u}}}\right] \qquad \text{but} \quad \varepsilon_{\mathrm{u}} \leq \varepsilon_{\mathrm{f}} \tag{5.14}$$

ε_{f} is the elongation after fracture as defined in the product standard

(3) f_{ua} is given by:

a) for austenitic and duplex stainless steels

$$f_{\mathrm{ua}} = \frac{f_{\mathrm{ya}}}{0{,}20 + 185\dfrac{f_{\mathrm{ya}}}{E}} \tag{5.15}$$

b) for ferritic stainless steels

$$f_{\mathrm{ua}} = \frac{f_{\mathrm{ya}}}{0{,}46 + 145\dfrac{f_{\mathrm{ya}}}{E}} \tag{5.16}$$

5.1.3 Fracture toughness

(1) Austenitic, ferritic and duplex stainless steels exhibit different fracture toughness characteristics.

NOTE Information on embrittlement due to contact with zinc in fire is given in A.5.

(2) The austenitic stainless steels covered in this document may be assumed to have sufficient fracture toughness to avoid brittle fracture of tension elements for service temperatures down to -50 °C.

If grades 1.4301, 1.4307, 1.4311, 1.4318 or 1.4306 are subject to extensive cold working, it should be demonstrated that the toughness of material greater than 10 mm thick is not less than the minimum *KV*-value specified in EN 10088-4 or EN 10088-5.

(3) If austenitic stainless steels are used at temperatures below -50 °C, tests should be carried out to demonstrate the material has a KV-value of at least 40 J at the lowest service temperature after the required fabrication steps.

(4) The ferritic stainless steels covered in this document may be assumed to have sufficient fracture toughness to avoid brittle fracture when the thickness of tension elements, t, is ≤ 5 mm. Where t > 5 mm, proof of sufficient fracture toughness should be obtained by applying the fracture mechanics concepts of EN 1993-1-10.

(5) For the duplex stainless steels covered in this document, Table 5.3 and Table 5.4 give the maximum permissible element thickness appropriate to a steel strength, its toughness requirement in terms of KV-value, the applied stress level σ_{Ed} and the reference temperature T_{Ed}. Table 5.3 is for Execution Class 3 and 4, and Table 5.4 for Execution Class 1 and 2.

As EN 10088 does not specify CVN requirements at low temperatures, Tables 5.3 and 5.4 specify maximum permissible values of element thickness for four toughness requirements, TR1 to TR4. The designer shall specify the minimum toughness requirement appropriate for the application.

σ_{Ed} and T_{Ed} should be calculated according to EN 1993-1-10. The values in the tables are based on the assumptions in EN 1993-1-10.

$f_{\mathrm{y}}(t) = f_{\mathrm{y}} - 0{,}25\ (t/t_0)$ with t_0 = 1 mm, or $f_{\mathrm{y}}(t)$ may be taken as $R_{\mathrm{p0,2}}$ from the product standard.

NOTE Tables 5.3 and 5.4 do not cover cold-formed sections and sheeting.

(6) If required, tests shall be carried out to demonstrate duplex stainless steels have a KV-value of at least 40 J at the test temperature for the selected quality in Table 5.3 or Table 5.4.

Table 5.3 — Maximum permissible values of element thickness t in mm for duplex stainless steel for Execution Class EXC3 and EXC4

Grade	f_y N/mm² (hot rolled plate)	Toughness requirement	KV		Reference Temperature T_{Ed} [°C]																				
			at T [°C]	J_{min} [J]	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
					$\sigma_{Ed} = 0,75 f_y(t)$							$\sigma_{Ed} = 0,50 f_y(t)$							$\sigma_{Ed} = 0,25 f_y(t)$						
1.4062, 1.4162, 1.4482	450	TR1	-20	40	70	60	45	40	30	25	20	120	100	85	70	55	45	40	185	160	140	120	105	90	75
		TR2	-30	40	85	70	60	45	40	30	25	140	120	100	85	70	55	45	200	185	160	140	120	105	90
1.4662	480	TR1	-20	40	65	55	45	35	30	20	15	110	95	80	65	55	45	35	180	155	135	115	100	85	70
		TR2	-30	40	80	65	55	45	35	30	20	130	110	95	80	65	55	45	200	180	155	135	115	100	85
1.4362	400	TR3	-40	40	110	95	80	65	50	40	35	175	150	130	110	90	75	65	200	200	195	170	150	130	110
		TR4	-50	40	130	110	95	80	65	50	40	200	175	150	130	110	90	75	200	200	200	195	170	150	130
1.4462	460	TR3	-40	40	100	85	70	55	45	35	30	160	135	115	95	80	70	55	200	200	185	160	140	120	100
		TR4	-50	40	120	100	85	70	55	45	35	185	160	135	115	95	80	70	200	200	200	185	160	140	120
1.4410, 1.4501, 1.4507	530	TR3	-40	40	90	75	60	50	40	30	25	145	125	105	85	70	60	50	200	195	170	150	125	110	95
		TR4	-50	40	110	90	75	60	50	40	30	170	145	125	105	85	70	60	200	200	195	170	150	125	110

NOTE 1 Linear interpolation can be used. Most applications require σ_{Ed} values between 0,75 $f_y(t)$ and 0,50 $f_y(t)$. σ_{Ed} = 0,25 $f_y(t)$ is given for interpolation purposes, and for use for elements under compression stress.

NOTE 2 Maximum permissible values of element thickness t are restricted to 200 mm. The values for thickness in the tables can exceed the upper thickness limit in the relevant product standards.

Table 5.4 — Maximum permissible values of element thickness t in mm for duplex stainless steel for Execution Class EXC1 and EXC2

Grade	f_y N/mm² (hot rolled plate)	Toughness require-ment	KV at T [°C]	KV J_{min} [J]	Reference Temperature T_{Ed} [°C]																				
					10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
					$\sigma_{Ed} = 0{,}75\, f_y(t)$							$\sigma_{Ed} = 0{,}50\, f_y(t)$							$\sigma_{Ed} = 0{,}25\, f_y(t)$						
1.4062, 1.4162, 1.4482	450	TR1	-20	40	200	200	200	150	105	75	50	200	200	200	200	200	195	140	200	200	200	200	200	200	200
		TR2	-30	40	200	200	200	200	150	105	75	200	200	200	200	200	200	195	200	200	200	200	200	200	200
1.4662	480	TR1	-20	40	200	200	195	135	95	65	45	200	200	200	200	200	175	125	200	200	200	200	200	200	200
		TR2	-30	40	200	200	200	195	135	95	65	200	200	200	200	200	200	175	200	200	200	200	200	200	200
1.4362	400	TR3	-40	40	200	200	200	200	200	185	130	200	200	200	200	200	200	200	200	200	200	200	200	200	200
		TR4	-50	40	200	200	200	200	200	200	185	200	200	200	200	200	200	200	200	200	200	200	200	200	200
1.4462	460	TR3	-40	40	200	200	200	200	200	145	100	200	200	200	200	200	200	200	200	200	200	200	200	200	200
		TR4	-50	40	200	200	200	200	200	200	145	200	200	200	200	200	200	200	200	200	200	200	200	200	200
1.4410, 1.4501, 1.4507	530	TR3	-40	40	200	200	200	200	160	110	80	200	200	200	200	200	200	200	200	200	200	200	200	200	200
		TR4	-50	40	200	200	200	200	200	160	110	200	200	200	200	200	200	200	200	200	200	200	200	200	200

NOTE 1 Linear interpolation can be used. Most applications require σ_{Ed} values between 0,75 $f_y(t)$ and 0,50 $f_y(t)$. σ_{Ed} = 0,25 $f_y(t)$ is given for interpolation purposes, and for use for elements under compression stress.

NOTE 2 Maximum permissible values of element thickness t are restricted to 200 mm. The values for thickness in the tables can exceed the upper thickness limit in the relevant product standards.

5.1.4 Through-thickness properties

(1) It is not necessary to specify material with improved through-thickness properties for austenitic and duplex stainless steels because they do not exhibit lamellar tearing.

(2) If ferritic stainless steel plate thicker than 20 mm is used in welded tee, cruciform or corner joints, guidance on the choice of through-thickness properties in EN 1993-1-10 should be followed. However, the low sulphur content of these stainless steels means that lamellar tearing is unlikely to occur.

5.1.5 Values of other material properties

(1) The material properties to be adopted in calculations should be taken as the following mean values for stainless steels:

Modulus of elasticity E = 200 000 N/mm²

Shear modulus $$G = \frac{E}{2\,(1+\nu)} \approx 77\,000 \text{ N/mm}^2$$

Poisson's ratio in elastic range $\nu = 0{,}3$

Coefficient of linear thermal expansion (for $T \leq 100$°C)	$\alpha_{\mathrm{T}} = 16 \times 10^{-6}$ per K for austenitic stainless steels
	$\alpha_{\mathrm{T}} = 13 \times 10^{-6}$ per K for duplex stainless steels
	$\alpha_{\mathrm{T}} = 10 \times 10^{-6}$ per K for ferritic stainless steels

(2) Stress-strain curves according to prEN 1993-1-14:2022 may be used to describe the material behaviour.

(3) The material parameter ε is defined as follows:

$$\varepsilon = \sqrt{\frac{235}{f_{\mathrm{y}}}} \tag{5.18}$$

(4) For calculating deflections in individual members, the secant modulus appropriate to the stress in the member at the serviceability limit state may be used, see 9.2(5).

5.2 Connecting devices

5.2.1 Fasteners

(1) Stainless steel bolts and nuts should conform with EN ISO 3506–1 and EN ISO 3506-2. Washers should be of stainless steel and should conform with EN ISO 7089 or EN ISO 7090, as appropriate. The corrosion resistance of the bolts should be equivalent to, or better than, the corrosion resistance of the parent metal.

(2) The nominal yield strength f_{yb} and ultimate tensile strength f_{ub} of stainless steel bolts should be obtained from Table 5.5.

Table 5.5 — Nominal values of f_{yb} and f_{ub} for stainless steel bolts

Material groups	Property class to EN ISO 3506	Yield strength f_{yb} N/mm^2	Ultimate tensile strength f_{ub} N/mm^2
Austenitic	50	210	500
Austenitic and duplex	70	450	700
Austenitic and duplex	80	600	800
Austenitic and duplex	100	800	1000

5.2.2 Preloaded bolts

(1) Bolting assemblies of property classes 80 and 100 made of austenitic and duplex stainless steel may be used as preloaded bolting assemblies in slip-resistant connections. Their suitability for preloading shall be demonstrated, and the tightening parameters shall be derived by bolting qualification procedure testing. The bolting procedure qualification testing shall specify the structural bolting assembly, tightening method, tightening parameters, preload losses and inspection requirements.

NOTE The National Annex can specify the bolting procedure qualification testing. CEN/TS XXXX, *Bolt tightening qualification procedure for stainless steel preloaded bolts* is under preparation.

5.2.3 Welding consumables

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

(2) In addition to the requirements of EN 1993-1-8, the welding electrodes should be capable of producing a weld with a corrosion resistance that is adequate for the service environment.

(3) If the corrosion resistance of the deposited metal and weld metal are not less than that of the material to be welded, the corrosion resistance of the welding electrodes may be assumed to be adequate.

6 Durability

(1) The procedure for selecting an appropriate stainless steel grade for the service environment shall be in accordance with Annex A.

7 Structural analysis

7.1 Structural modelling for analysis

(1) The provisions given in EN 1993-1-1:2022, Clause 7 should be applied to stainless steels, except where modified or superseded by the special provisions given in this document.

7.2 Global analysis

7.2.1 Consideration of second order effects

(1) Second order analysis is not required if the conditions (7.1) and (7.2) of EN 1993-1-1:2022 are satisfied.

NOTE The value of k_0 in Formula (7.1) of EN 1993-1-1:2022 is $1/\bar{\lambda}_0{}^2$ where $\bar{\lambda}_0$ is the limiting relative slenderness according to Table 8.3, unless the National Annex gives a different value.

7.3 Imperfections

7.3.1 Equivalent bow imperfection for global and member design

(1) For second order elastic analysis and second order plastic hinge analysis, the equivalent bow imperfection, e_0, of members for flexural buckling may be determined from EN 1993-1-1:2022, 7.3.3.1.

(2) For second order plastic zone analysis using either a bi-linear material model (see 7.4.3.3) or the non-linear material model (see 7.4.3.4) provided for stainless steel in prEN 1993-1-14:2022, 5.3.3, the equivalent bow imperfection, e_0, of members for flexural buckling may be determined according to Formula (7.1):

$$e_0 = \frac{\alpha L}{150} \tag{7.1}$$

where

L is the member length;

α is the imperfection factor, depending on the relevant buckling curve according to Table 8.3.

7.3.2 Imperfection based on elastic critical buckling modes

(1) For second order elastic analysis and second order plastic hinge analysis (see 7.4.3.2), the shape of the elastic critical buckling mode η_{cr} of the structure may be applied as a unique global and local imperfection, in accordance with 7.3.6 of EN 1993-1-1:2022. The amplitude of this imperfection should be determined from Formula (7.2):

$$e_{0,\mathrm{m}} = \alpha_{\eta,\mathrm{m}}(\bar{\lambda}_{\mathrm{m}} - \bar{\lambda}_0)\frac{M_{\mathrm{Rk,m}}}{N_{\mathrm{Rk,m}}} \tag{7.2}$$

where

m	is an index that denotes the critical cross-section of the frame structure or of the verified member. Index m indicates that the value or property belongs to the critical cross-section;
$\bar{\lambda}_{\mathrm{m}} = \sqrt{\dfrac{N_{\mathrm{Rk,m}}}{N_{\mathrm{cr,m}}}}$	is the relative slenderness of the member determined at the critical cross-section m;
$N_{\mathrm{cr,m}} = \alpha_{\mathrm{cr}} N_{\mathrm{Ed,m}}$	is the value of critical axial force in the cross-section m and also the critical axial force for the equivalent member;
α_{cr}	is the minimum force amplifier for the axial force configuration N_{Ed} in members to reach the elastic critical buckling load of the structure;
$\alpha_{\eta,\mathrm{m}}$	is the imperfection factor α for the relevant buckling curve at the critical cross-section m, see Table 8.3;
$\bar{\lambda}_0$	is the limiting relative slenderness, see Table 8.3;
$M_{\mathrm{Rk,m}}$	is the characteristic value of the moment resistance of the critical cross-section m, e.g. $M_{\mathrm{el,Rk,m}}$ or $M_{\mathrm{pl,Rk,m}}$ as relevant;
$N_{\mathrm{Rk,m}}$	is the characteristic value of resistance to axial force of the critical cross-section m.

7.4 Methods of analysis considering material non-linearities

7.4.1 General

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

a) Elastic global analysis (see 7.4.2); or

b) Plastic (hinge or zone) global analysis (see 7.4.3.1).

7.4.2 Elastic global analysis

(1) Elastic global analysis may be used for all structures where the response of all members contributing to the global stability remains predominantly elastic under all load cases (see 7.4.2(3)) and structures where loss of stiffness due to material non-linearity has a negligible effect on the internal forces.

(2) Material non-linearity reduces the stiffness of the structure. The effect of this reduction in stiffness should be considered by performing a plastic zone analysis (see 7.4.3.3 or 7.4.3.4) if it increases the action effects significantly or modifies the structural behaviour significantly.

(3) Members are deemed to remain predominantly elastic if condition (7.3) is satisfied.

$$\frac{E_{\mathrm{s}}}{E} > 0{,}2 \tag{7.3}$$

where E_{s} is the secant modulus corresponding to the stress σ, given by (7.4):

$$E_{\mathrm{s}} = \frac{E}{1+0{,}002\frac{E}{\sigma}\left(\frac{\sigma}{f_{\mathrm{y}}}\right)^{n}} \tag{7.4}$$

E is the modulus of elasticity;

σ is the maximum stress obtained from a first order elastic analysis in the cross-section of any member contributing to the global stability of the structure at the design load level;

f_y is the yield stress;

n is the strain hardening exponent, see Table 9.1.

7.4.3 Plastic global analysis

7.4.3.1 General

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

a) Plastic hinge method (see 7.4.3.2): the non-linear material behaviour is concentrated in plastified sections and/or joints as plastic hinges;

b) Plastic zone method with a bi-linear material model (see 7.4.3.3): the partial plastification of members in plastic zones is explicitly considered using an elastic, perfectly plastic material model (see EN 1993-1-1:2022, Figure 7.9); strain hardening is also allowed for by using an elastic linear hardening material model (see B.4 and Figure B.1);

c) Plastic zone method with the two-stage Ramberg-Osgood material model (see 7.4.3.4): the partial plastification, gradual loss of stiffness and strain hardening of members in plastic zones is explicitly considered using the material model given in prEN 1993-1-14:2022, 5.3.3.

(2) For plastic analysis (hinge or zone), first order theory may be used for the determination of the in-plane (sway) bending moments in accordance with EN 1993-1-1:2022, 7.2.1(5) when the criterion (7.5) is satisfied:

$$\alpha_{\mathrm{cr,sw,mod}} \geq 10 \tag{7.5}$$

where

$\alpha_{\mathrm{cr,sw,mod}}$ is the modified factor, determined using Formula (7.6), by which the design load would have to be increased to cause elastic instability in a global (sway) mode to account for the influence of plasticity on the sway stiffness of frame;

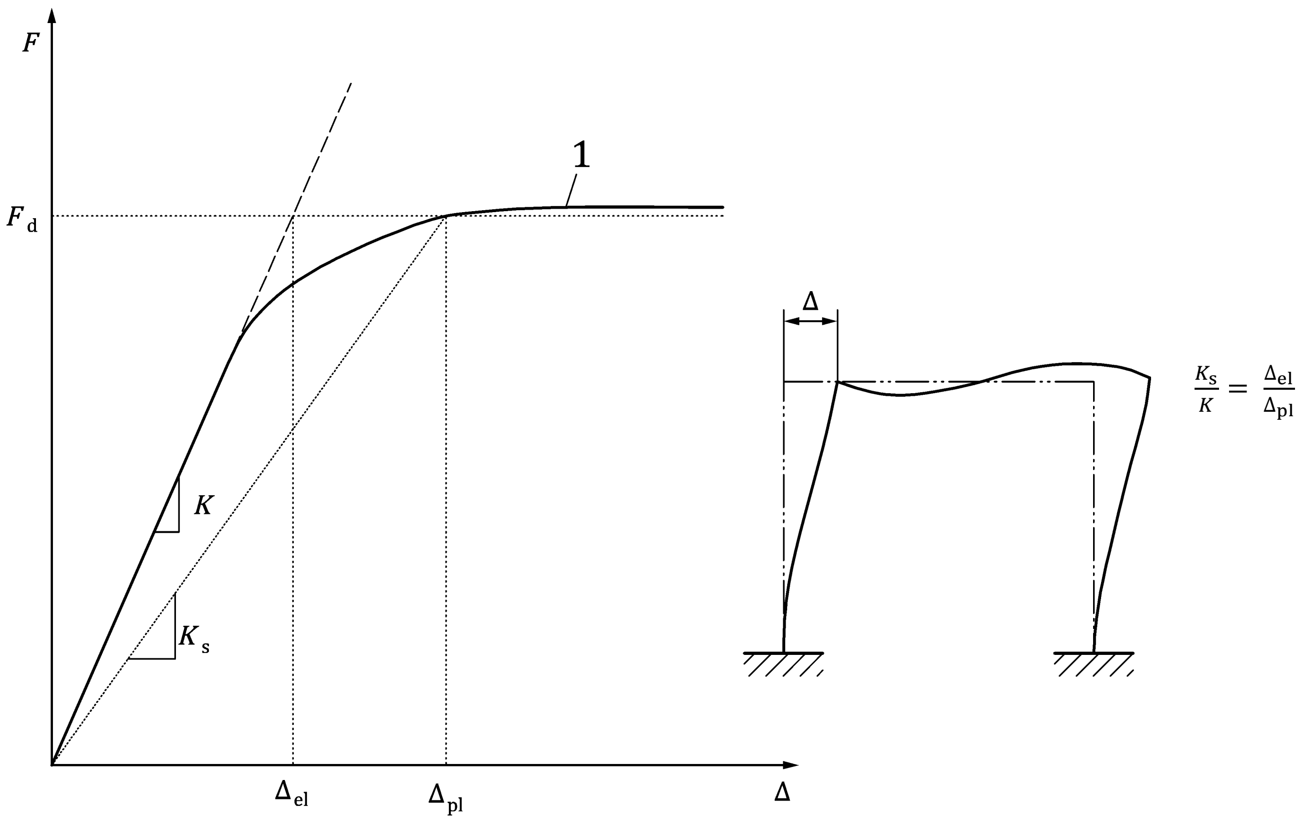
$$\alpha_{\mathrm{cr,sw,mod}} = \frac{K_{\mathrm{s}}}{K} Y \alpha_{\mathrm{cr,sw}} \tag{7.6}$$

K_{s}/K is the ratio of the secant lateral stiffness at the design value of the loading on the structure, F_{d}, to the initial lateral stiffness of the structure due to the influence of plasticity (i.e. as obtained from a first order plastic (hinge or zone) analysis, as illustrated in Figure 7.1); K_{s}/K may be alternatively expressed in terms of displacements as $\Delta_{\mathrm{el}}/\Delta_{\mathrm{pl}}$, as shown in Figure 7.1.

Y is the factor that approximates the further loss of stiffness due to second order effects, taken from Table 7.1.

$\alpha_{\mathrm{cr,sw}}$ may be calculated according to Formula (7.2) of EN 1993-1-1:2022.

For multi-storey frames, K_{s}/K should be calculated based on the critical storey.



Key

F load

Δ lateral displacement

1 Response obtained from a material nonlinear analysis

Figure 7.1 — Determination of ratio K_s/K

Table 7.1 — *Y* factor

Stainless steel	For single storey portal frames	For all other frames
Austenitic	0,80	0,55
Duplex	0,85	0,60
Ferritic	0,90	0,65

7.4.3.2 Plastic hinge method

(1) The requirements for plastic hinges set out in 7.4.3.1 of this document and EN 1993-1-1:2022, 7.6 should be followed.

(2) The effects of deformed geometry of the structure and the structural stability of the frame should be verified according to 7.4.3.1(2).

(3) The plastic hinge method may be used for austenitic and duplex stainless steels only.

7.4.3.3 Plastic zone method with a bi-linear material model

(1) The bi-linear elastic, perfectly-plastic stress-strain relationship indicated in EN 1993-1-1:2022, Figure 7.9 or the bi-linear elastic, linear hardening stress-strain relationship indicated in Figure B.1 may be used.

(2) The effects of deformed geometry of the structure and the structural stability of the frame should be verified according to 7.4.3.1(2).

(3) The verification of cross-section resistance may be performed either through cross-section resistance checks (see 8.2 or Annex B) or, more accurately, through the application of strain limits, see 7.4.3.5.

(4) Provided the continuous strength method (CSM) strain limits set out in 7.4.3.5 are satisfied, plastic zone analysis using a bi-linear material model may be applied to structures composed of all classes of cross-section. Cross-section classification is not required.

(5) In the application of methods M4 and M5 (see EN 1993-1-1:2022, 7.2.2) with plastic zone analysis using a bi-linear material model, the equivalent bow imperfections given in 7.3.1 should be used and the modulus of elasticity should be taken as the characteristic (fifth percentile) value equal to E = 191 000 N/mm^2.

7.4.3.4 Plastic zone method with the two stage Ramberg-Osgood material model

(1) The nonlinear material stress-strain response described by the two stage Ramberg-Osgood material model given in prEN 1993-1-14:2022 should be used.

(2) Provided the strain limits set out in 7.4.3.5 are satisfied, plastic zone analysis using the two stage Ramberg-Osgood material model may be applied to structures composed of all classes of cross-section. Cross-section classification is not required.

(3) In the application of methods M4 and M5 (see EN 1993-1-1:2022, 7.2.2) with plastic zone analysis using the two stage Ramberg-Osgood material model, the equivalent bow imperfections given in 7.3.1 should be used and the modulus of elasticity should be taken as the characteristic (fifth percentile) value equal to E = 191 000 N/mm^2.

7.4.3.5 Strain limits for plastic zone analysis

(1) Strain limits determined using the CSM, as given by Formulae (7.9) and (7.10), may be used for greater accuracy in place of cross-section checks for doubly-symmetric I-sections and rectangular hollow sections in plastic zone analysis. The strain limits simulate local buckling and control the extent to which spread of plasticity, moment redistribution and strain hardening are exploited.

(2) The overall structure or structural member shall satisfy criterion (7.7):

$$\frac{F_{\mathrm{d}}}{F_{\mathrm{Rd}}} \leq 1 \qquad (7.7)$$

where

$F_{\mathrm{Rd}} = F_{\mathrm{Rk}}/\gamma_{\mathrm{M1}}$

F_{Rk} is the characteristic value of the resistance of the structure, taken at the peak load factor attained during the plastic zone analysis or the point at which the CSM strain limit (see 7.4.3.5(3)) is reached, whichever is the lesser.

(3) The maximum longitudinal compressive strain at the design load level $\varepsilon_{\mathrm{Ed}}$ at each cross-section shall satisfy criterion (7.8):

$$\frac{\varepsilon_{\mathrm{Ed}}}{\varepsilon_{\mathrm{csm}}} \leq 1{,}0 \qquad (7.8)$$

where

$\varepsilon_{\mathrm{csm}}$ is the CSM strain limit given by Formula (7.9) when a bi-linear material model is used (see 7.4.3.3) and Formula (7.10) when the rounded two-stage Ramberg-Osgood material model is used (see 7.4.3.4)

$$\frac{\varepsilon_{\mathrm{csm}}}{\varepsilon_{\mathrm{y}}} = \begin{cases} \dfrac{0{,}25}{\bar{\lambda}_{\mathrm{p,cs}}^{\;3{,}6}} \leq \min\left(\Omega, \quad \dfrac{C_1 \varepsilon_{\mathrm{u}}}{\varepsilon_{\mathrm{y}}}\right) & \text{for } \bar{\lambda}_{\mathrm{p,cs}} \leq 0{,}68 \\ \left(1 - \dfrac{0{,}222}{\bar{\lambda}_{\mathrm{p,cs}}^{\;1{,}050}}\right) \dfrac{1}{\bar{\lambda}_{\mathrm{p,cs}}^{\;1{,}050}} & \text{for } 0{,}68 < \bar{\lambda}_{\mathrm{p,cs}} \leq 1{,}00 \end{cases} \tag{7.9}$$

$$\frac{\varepsilon_{\mathrm{csm}}}{\varepsilon_{\mathrm{y}}} = \begin{cases} \dfrac{0{,}25}{\bar{\lambda}_{\mathrm{p,cs}}^{\;3{,}6}} + \dfrac{0{,}002}{\varepsilon_{\mathrm{y}}} \leq \Omega & \text{for } \bar{\lambda}_{\mathrm{p,cs}} \leq 0{,}68 \\ \left(1 - \dfrac{0{,}222}{\bar{\lambda}_{\mathrm{p,cs}}^{\;1{,}050}}\right) \dfrac{1}{\bar{\lambda}_{\mathrm{p,cs}}^{\;1{,}050}} + \dfrac{0{,}002\left(\sigma / f_y\right)^n}{\varepsilon_{\mathrm{y}}} & \text{for } 0{,}68 < \bar{\lambda}_{\mathrm{p,cs}} < 1{,}00 \end{cases} \tag{7.10}$$

where

$\varepsilon_{\mathrm{y}} = f_{\mathrm{y}}/E$	is the elastic strain at the yield strength;
$\bar{\lambda}_{\mathrm{p,cs}} = \sqrt{f_{\mathrm{y}}/\sigma_{\mathrm{cr,cs}}}$	is the cross-section slenderness;
$\sigma_{\mathrm{cr,cs}}$	is the elastic local buckling stress of the full cross-section, which may be determined numerically or according to [11];
σ	is the maximum compressive stress;
C_1	is a material coefficient given in Table B.1;
Ω	is a project specific parameter that defines the maximum permissible level of plastic strain in the structure;
n	is the strain hardening exponent, see Table 9.1.

NOTE The value of Ω is 15 is unless the National Annex gives a different value.

(4) $\varepsilon_{\mathrm{Ed}}$ may be taken as the average value of the maximum strain over a length of member equal to the elastic local buckling half-wavelength $L_{\mathrm{b,cs}}$. $L_{\mathrm{b,cs}}$ may be determined numerically or according to [12]. A maximum element length equal to $L_{\mathrm{b,cs}}$ should be used. The strains should be averaged over the number of elements that lie wholly within $L_{\mathrm{b,cs}}$.

(5) The interaction between bending and shear should be accounted for by satisfying criterion (7.11):

$$\frac{\varepsilon_{\mathrm{Ed}}}{\rho_{\mathrm{csm}} \varepsilon_{\mathrm{csm}}} \leq 1 \tag{7.11}$$

where

$$\rho_{\mathrm{csm}} = \begin{cases} 1 & \text{for} \quad V_{\mathrm{Ed}} \leq 0{,}5 V_{\mathrm{pl,Rd}} \\ \dfrac{0{,}5}{0{,}5+\rho} & \text{for} \quad V_{\mathrm{Ed}} > 0{,}5 V_{\mathrm{pl,Rd}} \end{cases} \tag{7.12}$$

$$\rho = \left(\frac{2V_{\mathrm{Ed}}}{V_{\mathrm{pl,Rd}}} - 1\right)^2 \tag{7.13}$$

V_{Ed} is the design shear force;

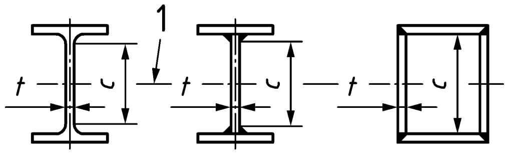
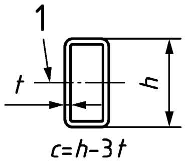
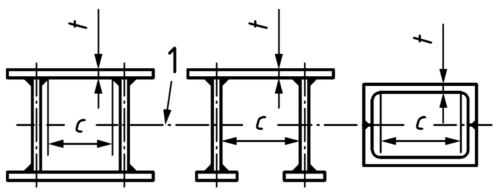
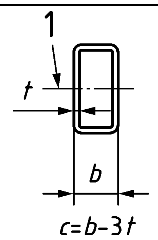
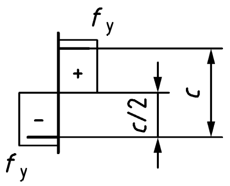
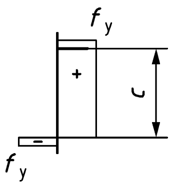
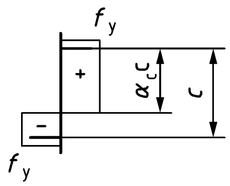
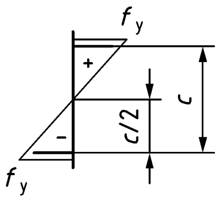
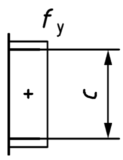
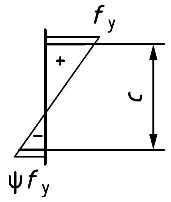
$V_{\mathrm{pl,Rd}}$ is the plastic shear resistance of the cross-section.

7.5 Classification of cross-sections

(1) The maximum width-to-thickness ratios for Class 1, 2, and 3 compression parts should be obtained from Table 7.2 to Table 7.4. They depend on the material parameter ε defined in 5.1.5(3). A part that fails to satisfy the limits for Class 3 should be taken as Class 4.

(2) For cold-formed sections and sheeting, if the resistance of the cross-section is to be determined based on the average yield strength f_{ya}, the classification of the cross-section shall also be based on the average yield strength by replacing f_{y} with f_{ya} in the definition of material parameter ε and parameter α_{c}.

Table 7.2 — Maximum width-to-thickness ratios for internal compression parts

Internal compression parts				
				
				
Key 1 Axis of bending				
		Part subject to bending	Part subject to compression	Part subject to bending and axial force
Stress distribution in parts (compression positive)	section type			
Class 1	All	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha_c > 0{,}5$: $c/t \leq \frac{396\varepsilon}{13\,\alpha_c - 1}$ when $\alpha_c \leq 0{,}5$: $c/t \leq \frac{36\varepsilon}{\alpha_c}$
Class 2	All	$c/t \leq 76\varepsilon$	$c/t \leq 35\varepsilon$	when $\alpha_c > 0{,}5$: $c/t \leq \frac{420\varepsilon}{13\,\alpha_c - 1}$ when $\alpha_c \leq 0{,}5$: $c/t \leq \frac{38\varepsilon}{\alpha_c}$
Stress distribution in parts (compression positive)	section type			
Class 3	welded	$c/t \leq 87\varepsilon$	$c/t \leq 35{,}4\varepsilon$	when $\psi > -1$: $c/t \leq \frac{35{,}4\varepsilon}{0{,}71 + 0{,}296\psi + 0{,}006\psi^2}$ when $\psi \leq -1$ [a]: $c/t \leq 43{,}5\varepsilon(1-\psi)$
	other	$c/t \leq 99\varepsilon$	$c/t \leq 37\varepsilon$	when $\psi > -1$: $c/t \leq \frac{37\varepsilon}{0{,}678 + 0{,}318\psi + 0{,}012\psi^2}$ when $\psi \leq -1$ [a]: $c/t \leq 49\varepsilon(1-\psi)$

For rectangular hollow sections and channels, I or H sections with equal flanges, under axial force and bending moment about the main axis parallel to the flanges, the parameter α_c that defines the position of the plastic neutral axis may be calculated as follows: If $N_{Ed} \geq c\, t_w n f_y$ $\alpha_c = 1{,}0$ If $N_{Ed} \leq -c\, n t_w f_y$ $\alpha_c = 0$ In all other cases: $\alpha_c = 0{,}5\left(1 + \frac{N_{Ed}}{c n t_w f_y}\right)$ Where N_{Ed} is the design axial force taken as positive for compression and negative for tension, and n = 1 for channels, I or H sections and n = 2 for rectangular hollow sections.
a $\psi \leq -1$ and a compression stress of $\sigma_{com,Ed} = f_y$ applies where the tensile strain $\varepsilon_t > f_y/E$.

Table 7.3 — Maximum width-to-thickness ratios for compression parts of outstand flanges

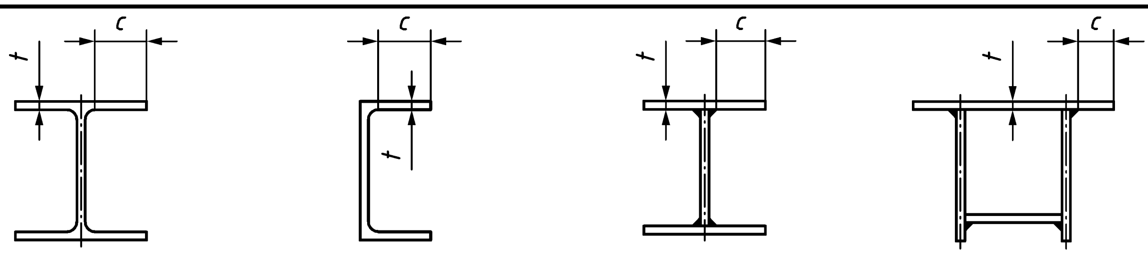
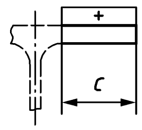
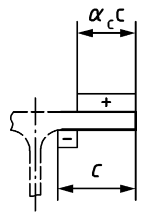
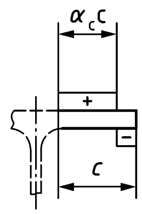
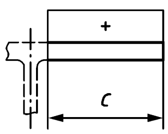
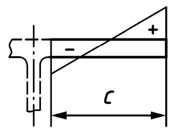
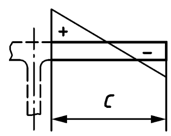
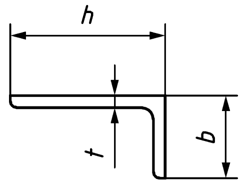
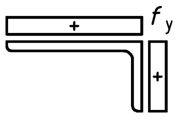
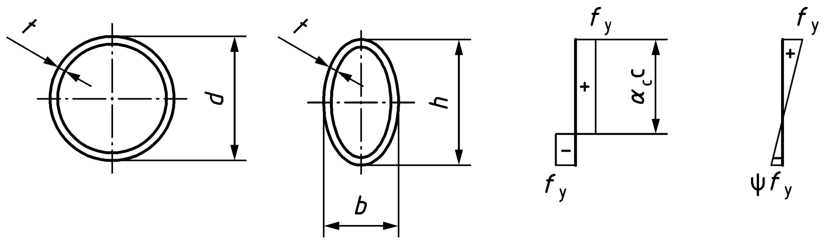
Outstand flanges				
 Rolled sections / Welded sections				
		Part in pure compression	Part subject to bending and axial force	
Stress distribution in parts (compression positive)	section type			
Class 1	All	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha_c}$	$c/t \leq \frac{9\varepsilon}{\alpha_c\sqrt{\alpha_c}}$
Class 2	All	$c/t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha_c}$	$c/t \leq \frac{10\varepsilon}{\alpha_c\sqrt{\alpha_c}}$
Stress distribution in parts (compression positive)	section type			
Class 3	welded	$c/t \leq 11{,}5\varepsilon$	$c/t \leq 17\varepsilon\sqrt{k_\sigma}$	
	other	$c/t \leq 14\varepsilon$	$c/t \leq 21{,}0\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

Table 7.4 — Maximum width-to-thickness ratios for compression parts of angles and circular and elliptical hollow sections

Angles		
Refer also to “Outstand flanges” (Table 7.3)		Does not apply to angles in continuous contact with other components

Stress distribution across section (compression positive)	
Class 3	$\frac{h}{t} \leq 15\varepsilon$ and $\frac{b+h}{2t} \leq 11{,}5\varepsilon$

Circular and elliptical hollow sections



	Section in compression	Section in bending	Section under bending and axial force
Class 1	$d_e/t \leq 50\varepsilon^2$	$d_e/t \leq 50\varepsilon^2$	$d_e/t \leq 50\varepsilon^2$
Class 2	$d_e/t \leq 70\varepsilon^2$	$d_e/t \leq 70\varepsilon^2$	$d_e/t \leq 70\varepsilon^2$
Class 3	$d_e/t \leq 90\varepsilon^2$ NOTE For $d_e/t > 90\varepsilon^2$, see 8.2.2(6).	$d_e/t \leq 280\varepsilon^2$ NOTE For $d_e/t > 280\varepsilon^2$, see EN 1993-1-6.	$d_e/t \leq \frac{5040\varepsilon^2}{19\psi + 37}$

Equivalent diameter d_e for circular and elliptical hollow sections
For circular hollow sections: $d_e = d$
For elliptical hollow sections: In compression: $d_e = d_{e,c} = h\left[1 + \left(1 - 2{,}3\left(\frac{t}{h}\right)^{0{,}6}\right)\left(\frac{h}{b} - 1\right)\right]$ or, conservatively: $d_e = d_{e,c} = \frac{h^2}{b}$ In bending about the strong axis: For $h/b \leq 1{,}36$: $d_e = d_{e,b,y} = \frac{b^2}{h}$ For $h/b > 1{,}36$: $d_e = d_{e,b,y} = 0{,}4\frac{h^2}{b}$ In bending about the weak axis, or compression and bending about the weak axis: $d_e = d_{e,b,z} = \frac{h^2}{b}$

In compression and bending about the strong axis, d_e may be determined by linear interpolation between $d_{e,c}$ and $d_{e,b,y}$ as given by:

$d_e = d_{e,b,y} + (d_{e,c} - d_{e,b,y})(2\alpha_c - 1)$ for Class 1 and 2 cross-sections

$d_e = d_{e,b,y} + (d_{e,c} - d_{e,b,y})(\psi + 1)/2$ for Class 3 and 4 cross-sections

In compression and biaxial bending, d_e may be taken as described above, but with α_c and ψ determined using a modified axial force equal to $N_{Ed} + M_{z,Ed}\, A/W_{pl,z}$ for Class 1 and Class 2 cross-sections and $N_{Ed} + M_{z,Ed}\, A/W_{el,z}$ for Class 3 and Class 4 cross-sections.

8 Ultimate limit states

8.1 Partial factors

(1) The following partial factors γ_{Mi} as defined in EN 1993-1-1:2022, 8.1(1) and EN 1993-1-8:2022, 4.3.2(1) should be applied to the characteristic values of the following resistances:

resistance of cross-sections (whatever the class is): γ_{M0}

resistance of members to instability assessed by member checks: γ_{M1}

resistance of cross-sections in tension to fracture: γ_{M2}

resistance of bolts, pins, welds and plates in bearing: γ_{M2}

— slip resistance:

at ultimate limit state (Category C in accordance with EN 1993-1-8) γ_{M3}

at serviceability limit state (Category B in accordance with EN 1993-1-8) $\gamma_{M3,ser}$

NOTE The partial factors γ_{M0}, γ_{M1}, γ_{M2}, γ_{M3} and $\gamma_{M3,ser}$ for stainless steel structural elements designed in accordance with any of the parts of EN 1993 are given below unless the National Annex of prEN 1993-1-4 gives different values:

$\gamma_{M0} = 1{,}10$

$\gamma_{M1} = 1{,}10$

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

$\gamma_{M3} = 1{,}25$

$\gamma_{M3,ser} = 1{,}10$

8.2 Resistance of cross-sections

8.2.1 General

(1) The provisions for the resistance of cross-sections given in EN 1993-1-1, EN 1993-1-3 and EN 1993-1-5, as appropriate, should be applied to stainless steels except as supplemented or modified in 8.2.2, 8.2.3, 8.2.4, 8.2.5, 8.2.6 or 8.2.7.

(2) Alternatively, the CSM provisions given in Annex B may be used to account for partial plastification and strain hardening in the determination of the cross-section resistance of I-sections, channels, T-sections, angles and rectangular and circular hollow sections subject to tension, compression, bending moment, and combined bending and axial force.

(3) The provisions in EN 1993-1-1:2022, Annex B for the design of semi-compact sections shall not be applied to stainless steel.

8.2.2 Effective cross-section properties

(1) The reduction in resistance due to the effects of local buckling in Class 4 cross-sections and distortional buckling in cross-sections with longitudinal stiffeners may be taken into account by using effective cross-section properties.

(2) The effective properties of Class 4 cross-sections should be based on the effective widths of the compression parts.

(3) For cold-formed sections (other than structural hollow sections) and sheeting, the effective cross-section properties should be determined following the procedure specified in prEN 1993-1-3:2022, 7.6. The effective widths should be determined according to (5) of this document. When calculating effective widths of the compressed elements or the reduced thicknesses of the stiffeners, the yield strength may be replaced by the average yield strength f_{ya} determined in accordance with 5.1.2.3.

(4) For structural hollow sections, the effective widths should be determined according to (5) and the yield strength may be replaced by the average yield strength f_{ya} determined in accordance with 5.1.2.3.

(5) The effective widths of planar compression parts should be determined according to EN 1993-1-5, except that the reduction factor ρ should be taken as follows:

For welded sections:

Internal compression elements

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} \le 0{,}328 + \sqrt{0{,}100 - 0{,}003\psi}$$

$$\rho = \frac{0{,}655\,\bar{\lambda}_{\mathrm{p}} - 0{,}003(3+\psi)}{\bar{\lambda}_{\mathrm{p}}^2} \le 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} > 0{,}328 + \sqrt{0{,}100 - 0{,}003\psi} \tag{8.1}$$

Outstand compression elements

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} \le 0{,}639$$

$$\rho = \frac{0{,}655\,\bar{\lambda}_{\mathrm{p}} - 0{,}01}{\bar{\lambda}_{\mathrm{p}}^2} \le 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} > 0{,}639 \tag{8.2}$$

For other sections:

Internal compression elements

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} \leq 0{,}386 + \sqrt{0{,}089 - 0{,}02\psi}$$

$$\rho = \frac{0{,}772\,\bar{\lambda}_{\mathrm{p}} - 0{,}02(3+\psi)}{\bar{\lambda}_{\mathrm{p}}^{2}} \leq 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} > 0{,}386 + \sqrt{0{,}089 - 0{,}02\psi} \tag{8.3}$$

Outstand compression elements

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} \leq 0{,}748$$

$$\rho = \frac{\bar{\lambda}_{\mathrm{p}} - 0{,}188}{\bar{\lambda}_{\mathrm{p}}^{2}} \leq 1{,}0 \qquad \text{for } \bar{\lambda}_{\mathrm{p}} > 0{,}748 \tag{8.4}$$

where $\bar{\lambda}_p$ is the element slenderness defined as:

$$\bar{\lambda}_{\mathrm{p}} = \sqrt{\frac{f_{\mathrm{y}}}{\sigma_{\mathrm{cr,p}}}} = \frac{\bar{b}/t}{27{,}7\varepsilon\sqrt{k_{\sigma}}} \tag{8.5}$$

ψ is the stress ratio determined in accordance with prEN 1993-1-5:2022, 6.4.1(3) and 6.4.1(4)

$\bar{b}$ is the appropriate width to be taken as follows (for definitions, see Table 7.2 to Table 7.4)

c for internal or outstand flat elements (except angles);

c for webs and flanges of rolled rectangular hollow sections, which can conservatively be taken as h-3t or b-3t, as appropriate;

h for equal-leg and unequal-leg angles;

k_{σ} is the buckling factor corresponding to the stress ratio ψ and boundary conditions from prEN 1993-1-5:2022, Table 6.1 or Table 6.2, as appropriate

t is the thickness of the plate

$\sigma_{\mathrm{cr,p}}$ is the elastic critical plate buckling stress;

ε is the material factor defined in 5.1.5(3).

(6) For Class 4 circular or elliptical hollow sections in compression conforming to EN 10296-2 or EN 10297-2, the effective cross-sectional area A_{eff} should be determined using the equivalent diameter d_{e} and the thickness t as:

$$A_{\mathrm{eff}} = A\sqrt{\frac{90\varepsilon^{2}}{d_{\mathrm{e}}/t}} \qquad \text{for } d_{\mathrm{e}}/t \leq 250\varepsilon^{2} \tag{8.6}$$

NOTE For Class 4 circular or elliptical hollow sections in compression exceeding the limits of d_{e}/t specified in (6), see EN 1993-1-6.

8.2.3 Compression

(1) The design resistance of the cross-section for uniform compression $N_{c,Rd}$ should be taken as:

$$N_{c,Rd} = \frac{N_{Rk}}{\gamma_{M0}} \tag{8.7}$$

where

$N_{Rk} = A\, f_y$	For Class 1, 2 or 3 cross-sections
$N_{Rk} = A_{eff}\, f_y$	For Class 4 cross-sections

(2) For cold-formed sections (including structural hollow sections) and sheeting, f_y in 8.2.3(1) may be replaced by f_{ya}.

8.2.4 Bending

(1) The design bending moment resistance of the cross-section for bending about one principal axis $M_{c,Rd}$ should be taken, as:

$$M_{c,Rd} = \frac{M_{Rk}}{\gamma_{M0}} \tag{8.8}$$

where

$M_{Rk} = W_{pl}\, f_y$	For Class 1 or 2 cross-sections
$M_{Rk} = W_{el}\, f_y$	For Class 3 cross-sections
$M_{Rk} = W_{eff}\, f_y$	For Class 4 cross-sections

(2) For cold-formed sections (including structural hollow sections) and sheeting, f_y in 8.2.4(1) may be replaced by f_{ya}. In addition, provided that the bending moment is applied about only one principal axis of the cross-section, and provided that yielding occurs first at the tension edge, the plastic reserve capacity in the tension zone may be utilised according to prEN 1993-1-3:2022, 8.1.4.2 until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{ya}/γ_{M0}.

8.2.5 Shear

(1) The design shear resistance $V_{c,Rd}$ of welded or hot rolled sections should be taken as the lesser of the shear buckling resistance $V_{b,Rd}$ according to EN 1993-1-5 modified by (3) and (4), and the elastic or plastic shear resistance according to EN 1993-1-1. For cold-formed sections, see prEN 1993-1-3:2022.

(2) Unstiffened webs with h_w/t greater than $\frac{56,2}{\eta}\varepsilon$, or stiffened webs with h_w/t greater $\frac{24,3}{\eta}\varepsilon\sqrt{k_\tau}$, should be checked for resistance to shear buckling and should be provided with transverse stiffeners at the supports.

where

h_w is the clear web depth between flanges, see prEN 1993-1-5:2022, Figure 7.1

ε is defined in 5.1.5(3)

k_τ is defined in prEN 1993-1-5:2022, 7.3

η = 1,20

(3)For webs with transverse stiffeners at supports only and for webs with either intermediate transverse or longitudinal stiffeners or both, the factor χ_{w} for the contribution of the web to the shear buckling resistance should be obtained from Table 8.1. The factor χ_{w} according to Table 8.1 is only valid for slendernesses $\bar{\lambda}_{\mathrm{w}}$ which are determined for plates with hinged boundary conditions.

Table 8.1 — Web shear buckling reduction factor χ_{w}

	End panels with non-rigid end post	**All the other cases (intermediate panels and end panels with rigid end post)**
$\bar{\lambda}_{\mathrm{w}} \leq 0{,}65/\eta$	η	η
$0{,}65/\eta < \bar{\lambda}_{\mathrm{w}} < 0{,}65$	$0{,}65/\bar{\lambda}_{\mathrm{w}}$	$0{,}65/\bar{\lambda}_{\mathrm{w}}$
$\bar{\lambda}_{\mathrm{w}} \geq 0{,}65$	$1{,}19/\left(0{,}54+\bar{\lambda}_{\mathrm{w}}\right)$	$1{,}56/\left(0{,}91+\bar{\lambda}_{\mathrm{w}}\right)$

End support conditions and $\bar{\lambda}_{\mathrm{w}}$ are defined in EN 1993-1-5.

(4) If the flange resistance is not fully utilised in withstanding the bending moment, i.e. $M_{\mathrm{Ed}} < M_{\mathrm{f,Rd}}$, then the contribution from the flanges $V_{\mathrm{bf,Rd}}$ may be calculated according to prEN 1993-1-5:2022, 7.4(1) but with c taken as:

$$c = \left[0{,}17 + \frac{3{,}5\, b_{\mathrm{f}}\, t_{\mathrm{f}}^{2}\, f_{\mathrm{yf}}}{t_{\mathrm{w}}\, h_{\mathrm{w}}^{2}\, f_{\mathrm{yw}}}\right] a \quad \text{and} \quad \frac{c}{a} \leq 0{,}65 \tag{8.9}$$

where

b_{f}, t_{f} and a are defined in EN 1993-1-5

f_{yf} is the yield strength of the flange;

f_{yw} is the yield strength of the web.

8.2.6 Resistance to transverse forces

(1) The design resistance of the webs of rolled beams and welded girders without web stiffeners should be determined in accordance with prEN 1993-1-5:2022, 8.2 modified by (2), provided that the compression flange is adequately restrained in the lateral direction.

(2) Values for α_{F0} and $\bar{\lambda}_{\mathrm{F0}}$ should be obtained from Table 8.2.

Table 8.2 — Values of α_{F0} and $\bar{\lambda}_{\mathrm{F0}}$

Load type (see prEN 1993-1-5:2022, 8.1(2))	**Austenitic and Duplex Stainless Steel**		**Ferritic Stainless Steel**	
	α_{F0}	$\bar{\lambda}_{\mathrm{F0}}$	α_{F0}	$\bar{\lambda}_{\mathrm{F0}}$
Type (a)	0,60	0,60	0,30	0,65
Type (b)				
Type (c)	0,75	0,50	0,75	0,50

8.2.7 Transverse web stiffeners

(1) The provisions in prEN 1993-1-5:2022, Clause 11 apply, except as modified by (2).

(2) When checking the buckling resistance, the effective cross-sectional area of a stiffener should include the stiffener itself plus a width of web of $11.5\varepsilon t_{\mathrm{w}}$ on each side of the stiffener, avoiding any overlap of contributing parts to adjacent stiffeners. At the ends of the member, the contributory width to be taken into account should be either $11.5\varepsilon t_{\mathrm{w}}$ or the nominal available width, whichever is the smaller.

8.3 Buckling resistance of members

8.3.1 General

(1) The provisions for flexural, lateral-torsional, torsional and torsional-flexural buckling given in EN 1993-1-1 and EN 1993-1-3, as appropriate, should be applied for stainless steels except as supplemented or modified in 8.3.2, 8.3.3 or 8.3.4.

(2) The provisions in EN 1993-1-1:2022, Annex B for the design of semi-compact sections shall not be applied to stainless steel.

8.3.2 Uniform members in compression

8.3.2.1 Buckling reduction factor for flexural buckling

(1) For flexural buckling of members in axial compression, the value of the buckling reduction factor χ for the appropriate relative slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to Formula (8.10):

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \qquad \text{but } \chi \leq 1{,}0 \tag{8.10}$$

where

$$\Phi = 0{,}5\left(1 + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \bar{\lambda}^2\right) \tag{8.11}$$

α is an imperfection factor, obtained from Table 8.3

$\bar{\lambda}_0$ is the limiting relative slenderness, obtained from Table 8.3

(2) The relative slenderness $\bar{\lambda}$ should be taken from Formula (8.12):

$$\bar{\lambda} = \sqrt{\frac{N_{\mathrm{Rk}}}{N_{\mathrm{cr}}}} \tag{8.12}$$

where

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross-section properties:

$N_{\mathrm{cr}} = N_{\mathrm{cr,y}}$ for elastic flexural buckling about the major axis, leading to $\bar{\lambda}_{\mathrm{y}}$;

$N_{\mathrm{cr}} = N_{\mathrm{cr,z}}$ for elastic flexural buckling about the minor axis, leading to $\bar{\lambda}_{\mathrm{z}}$;

$N_{\mathrm{cr}} = N_{\mathrm{cr,T}}$ for torsional buckling, leading to $\bar{\lambda}_{\mathrm{T}}$;

$N_{\mathrm{cr}} = N_{\mathrm{cr,TF}}$ for elastic torsional-flexural buckling about y-y, leading to $\bar{\lambda}_{\mathrm{TF}}$;

N_{Rk} is the characteristic value of the resistance to compression.

(3) Buckling effects may be ignored and only cross sectional checks apply when the relative slenderness $\bar{\lambda} \leq \bar{\lambda}_0$.

Table 8.3 — Values of α and $\bar{\lambda}_0$ for flexural buckling

Type of member	Axis of buckling	Austenitic		Duplex		Ferritic	
		α	$\bar{\lambda}_0$	α	$\bar{\lambda}_0$	α	$\bar{\lambda}_0$
Hot rolled and welded I-sections	Major	0,60	0,2	0,49	0,3	0,49	0,2
	Minor	0,76	0,2	0,60	0,3	0,76	0,2
Cold-formed rectangular hollow sections	Any	0,49	0,3	0,49	0,3	0,49	0,2
Cold-formed circular and elliptical hollow sections	Any	0,49	0,2	0,49	0,3	0,49	0,2
Hot finished rectangular hollow sections	Any	0,49	0,2	0,49	0,2	0,34	0,2
Hot finished circular and elliptical hollow sections	Any	0,49	0,2	0,49	0,2	0,34	0,2
Welded (including laser welded) box sections	Any	0,49	0,2	0,49	0,2	0,49	0,2
Cold-formed lipped channels and hat sections	Any	0,49	0,2	0,49	0,2	0,49	0,2
Other open sections	Any	0,76	0,2	0,76	0,2	0,76	0,2
For austenitic laser welded I-sections, α may be taken as 0,60 for buckling about the minor axis.							

8.3.2.2 Buckling reduction factor for torsional and torsional-flexural buckling

(1) For symmetric or asymmetric cross-sections, the buckling reduction factor χ_{T} for torsional buckling or χ_{TF} for torsional-flexural buckling may be determined by using Formulae (8.10) and (8.11), with the buckling coefficients for buckling about the minor axis in accordance with Table 8.3. The relative slenderness $\bar{\lambda}$ should be replaced by $\bar{\lambda}_{\mathrm{T}}$ or $\bar{\lambda}_{\mathrm{TF}}$ determined using Formula (8.12) based on the elastic torsional buckling force $N_{\mathrm{cr,T}}$, or the elastic torsional-flexural buckling force $N_{\mathrm{cr,TF}}$, respectively.

8.3.3 Uniform members in bending

8.3.3.1 Buckling reduction factors for lateral torsional buckling

(1) In general cases of prismatic members with arbitrary boundary conditions, the buckling reduction factor χ_{LT} may be determined by using Formulae (8.10) and (8.11), with the relative slenderness $\bar{\lambda}$ replaced by the relative slenderness for lateral torsional buckling $\bar{\lambda}_{\mathrm{LT}}$ from Formula (8.13), and with the limiting slenderness $\bar{\lambda}_0$ and the imperfection factor α taken as follows:

$$\bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{M_{\mathrm{Rk}}}{M_{\mathrm{cr}}}} \tag{8.13}$$

$\bar{\lambda}_0$ = 0,2

α = 0,34 for cold-formed sections and hollow sections (welded and seamless)

= 0,76 for welded open sections and other sections for which no test data are available

where

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

M_{Rk} is the characteristic value of the resistance to bending moment.

(2) For doubly symmetric I- and H-sections and fork boundary conditions at both ends, the buckling reduction factor χ_{LT} may be taken as:

$$\chi_{\mathrm{LT}} = \frac{f_{\mathrm{M}}}{\Phi_{\mathrm{LT}} + \sqrt{\Phi_{\mathrm{LT}}^{2} - f_{\mathrm{M}}\bar{\lambda}_{\mathrm{LT}}^{2}}} \quad \text{but } \chi_{\mathrm{LT}} \leq 1{,}0 \tag{8.14}$$

where

$$\Phi_{\mathrm{LT}} = 0{,}5\left[1 + f_{\mathrm{M}}\left(\left(\frac{\bar{\lambda}_{\mathrm{LT}}}{\bar{\lambda}_{\mathrm{z}}}\right)^{2} \alpha_{\mathrm{LT}}\left(\bar{\lambda}_{\mathrm{z}} - 0{,}2\right) + \bar{\lambda}_{\mathrm{LT}}^{2}\right)\right] \tag{8.15}$$

α_{LT} is the imperfection factor taken from Table 8.4;

$\bar{\lambda}_{\mathrm{z}}$ is the corresponding relative slenderness for weak axis flexural buckling, as defined in 8.3.2.1, with the buckling length $L_{\mathrm{cr,z}}$, taken as the distance between the discrete lateral restraints;

f_{M} is a factor that accounts for the effect of the bending moment distribution between discrete lateral restraints. It may conservatively be taken as 1,0 in cases that cannot be approximated by the diagrams in EN 1993-1-1:2022, Table 8.6.

(3) Lateral torsional buckling effects may be ignored and only cross sectional checks apply when the relative slenderness $\bar{\lambda}_{\mathrm{LT}} \leq 0{,}4$.

Table 8.4 — Imperfection factor α_{LT} for lateral torsional buckling of doubly symmetric I- and H-sections

Steel grade	**Imperfection factor α_{LT}**
Austenitic	$0{,}31\sqrt{\frac{W_{\mathrm{el,y}}}{W_{\mathrm{el,z}}}}$ but: $\alpha_{\mathrm{LT}} \leq 1{,}10$
Duplex	$0{,}23\sqrt{\frac{W_{\mathrm{el,y}}}{W_{\mathrm{el,z}}}}$ but: $\alpha_{\mathrm{LT}} \leq 0{,}76$
Ferritic	$0{,}27\sqrt{\frac{W_{\mathrm{el,y}}}{W_{\mathrm{el,z}}}}$ but: $\alpha_{\mathrm{LT}} \leq 0{,}76$

8.3.4 Uniform members in bending and axial compression

(1) The stability of members, including cold-formed section members, should be verified using the rules given in EN 1993-1-1:2022, 8.3.3, except as modified or superseded by (2) and (3).

(2) The provisions in EN 1993-1-1:2022, C.1 for the design of mono-symmetric I- , H- and welded box section members shall not be applied to stainless steel.

(3) Members which are subjected to combined bending and axial compression should satisfy the interaction formulae given in EN 1993-1-1:2022, 8.3.3(5), but using the interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} given in Table 8.5 and Table 8.6.

Table 8.5 — Interaction factors k_{yy} and k_{yz} for EN 1993-1-1:2022, criterion (8.88), Instability governed by buckling about y-y axis

Austenitic	Duplex	Ferritic
I-sections		
For $\bar{\lambda}_y < 1{,}0$: $k_{yy} = C_{my}[1 + 2{,}50(\bar{\lambda}_y - 0{,}35)n_y]$	For $\bar{\lambda}_y < 1{,}3$: $k_{yy} = C_{my}[1 + 2{,}00(\bar{\lambda}_y - 0{,}30)n_y]$	For $\bar{\lambda}_y < 1{,}3$: $k_{yy} = C_{my}[1 + 1{,}60(\bar{\lambda}_y - 0{,}35)\mathrm{n}_y]$
For $\bar{\lambda}_y \geq 1{,}0$: $k_{yy} = C_{my}(1 + 1{,}625n_y)$	For $\bar{\lambda}_y \geq 1{,}3$: $k_{yy} = C_{my}(1 + 2{,}00n_y)$	For $\bar{\lambda}_y \geq 1{,}3$: $k_{yy} = C_{my}(1 + 1{,}52n_y)$
$k_{yz} = k_{zz}$ (for k_{zz} see Table 8.6)		
Rectangular hollow sections and welded box sections		
For $\bar{\lambda}_y < 1{,}3$: $k_{yy} = C_{my}[1 + 2{,}00(\bar{\lambda}_y - 0{,}30)n_y]$	For $\bar{\lambda}_y < 1{,}4$: $k_{yy} = C_{my}[1 + 1{,}50(\bar{\lambda}_y - 0{,}40)n_y]$	For $\bar{\lambda}_y < 1{,}6$: $k_{yy} = C_{my}[1 + 1{,}30(\bar{\lambda}_y - 0{,}45)n_y]$
For $\bar{\lambda}_y \geq 1{,}3$: $k_{yy} = C_{my}(1 + 2{,}00n_y)$	For $\bar{\lambda}_y \geq 1{,}4$: $k_{yy} = C_{my}(1 + 1{,}5n_y)$	For $\bar{\lambda}_y \geq 1{,}6$: $k_{yy} = C_{my}(1 + 1{,}495\mathrm{n}_y)$
$k_{yz} = k_{zz}$ (for k_{zz} see Table 8.6)		
Circular hollow sections		
For $\bar{\lambda}_y < 1{,}3$: $k_{yy} = C_{my}[1 + 2{,}50(\bar{\lambda}_y - 0{,}30)n_y]$	For $\bar{\lambda}_y < 1{,}3$: $k_{yy} = C_{my}[1 + 2{,}00(\bar{\lambda}_y - 0{,}38)n_y]$	For $\bar{\lambda}_y < 1{,}3$: $k_{yy} = C_{my}[1 + 1{,}90(\bar{\lambda}_y - 0{,}39)n_y]$
For $\bar{\lambda}_y \geq 1{,}3$: $k_{yy} = C_{my}(1 + 2{,}50n_y)$	For $\bar{\lambda}_y \geq 1{,}3$: $k_{yy} = C_{my}(1 + 1{,}84n_y)$	For $\bar{\lambda}_y \geq 1{,}3$: $k_{yy} = C_{my}(1 + 1{,}805n_y)$
$k_{yz} = k_{yy}$		
NOTE 1 See EN 1993-1-1:2022, 8.3.3(9) for n_y.		
NOTE 2 See EN 1993-1-1:2022, 8.3.3(10) and EN 1993-1-1:2022, Table 8.9 for C_{my}.		

Table 8.6 — Interaction factors k_{zy} and k_{zz} for EN 1993-1-1:2022, criterion (8.89), Instability governed by buckling about z-z axis

Austenitic	Duplex	Ferritic
I-sections		
For $\bar{\lambda}_z < 0,8$: $k_{zy} = 1 - \frac{0,2\bar{\lambda}_z n_z}{C_{mLT} - 0,4}$	For $\bar{\lambda}_z < 0,8$: $k_{zy} = 1 - \frac{0,2\bar{\lambda}_z n_z}{C_{mLT} - 0,4}$	For $\bar{\lambda}_z < 0,8$: $k_{zy} = 1 - \frac{0,2\bar{\lambda}_z n_z}{C_{mLT} - 0,4}$
For $\bar{\lambda}_z \geq 0,8$: $k_{zy} = 1 - \frac{0,16 n_z}{C_{mLT} - 0,4}$	For $\bar{\lambda}_z \geq 0,8$: $k_{zy} = 1 - \frac{0,16 n_z}{C_{mLT} - 0,4}$	For $\bar{\lambda}_z \geq 0,8$: $k_{zy} = 1 - \frac{0,16 n_z}{C_{mLT} - 0,4}$
For $\bar{\lambda}_z < 1,2$: $k_{zz} = C_{mz}\left[1 + 2,80\left(\bar{\lambda}_z - 0,50\right)n_z\right]$	For $\bar{\lambda}_z < 1,2$: $k_{zz} = C_{mz}\left[1 + 2,70\left(\bar{\lambda}_z - 0,50\right)n_z\right]$	For $\bar{\lambda}_z < 1,5$: $k_{zz} = C_{mz}\left[1 + 2,20\left(\bar{\lambda}_z - 0,50\right)n_z\right]$
For $\bar{\lambda}_z \geq 1,2$: $k_{zz} = C_{mz}(1 + 1,96 n_z)$	For $\bar{\lambda}_z \geq 1,2$: $k_{zz} = C_{mz}(1 + 1,89 n_z)$	For $\bar{\lambda}_z \geq 1,5$: $k_{zz} = C_{mz}(1 + 2,20 n_z)$
Rectangular hollow sections and welded box sections		
$k_{zy} = k_{yy}$ (for k_{yy} see Table 8.5)		
For $\bar{\lambda}_z < 1,3$: $k_{zz} = C_{mz}\left[1 + 2,00\left(\bar{\lambda}_z - 0,30\right)n_z\right]$	For $\bar{\lambda}_z < 1,4$: $k_{zz} = C_{mz}\left[1 + 1,50\left(\bar{\lambda}_z - 0,40\right)n_z\right]$	For $\bar{\lambda}_z < 1,6$: $k_{zz} = C_{mz}\left[1 + 1,30\left(\bar{\lambda}_z - 0,45\right)n_z\right]$
For $\bar{\lambda}_z \geq 1,3$: $k_{zz} = C_{mz}(1 + 2,00 n_z)$	For $\bar{\lambda}_z \geq 1,4$: $k_{zz} = C_{mz}(1 + 1,50 n_z)$	For $\bar{\lambda}_z \geq 1,6$: $k_{zz} = C_{mz}(1 + 1,495 n_z)$
Circular hollow sections		
$k_{zz} = k_{zy} = k_{yy}$ (for k_{yy} see Table 8.5)		
Cold-formed and hot rolled channels		
For $\bar{\lambda}_z < 0,9$: $k_{zz} = C_{mz}\left[1 + 5,00\left(\bar{\lambda}_z - 0,20\right)n_z\right]$	For $\bar{\lambda}_z < 1,3$: $k_{zz} = C_{mz}\left[1 + 2,50\left(\bar{\lambda}_z - 0,40\right)n_z\right]$	For $\bar{\lambda}_z < 1,3$: $k_{zz} = C_{mz}\left[1 + 2,50\left(\bar{\lambda}_z - 0,40\right)n_z\right]$
For $\bar{\lambda}_z \geq 0,90$: $k_{zz} = C_{mz}(1 + 3,50 n_z)$	For $\bar{\lambda}_z \geq 1,30$: $k_{zz} = C_{mz}(1 + 2,25 n_z)$	For $\bar{\lambda}_z \geq 1,30$: $k_{zz} = C_{mz}(1 + 2,25 n_z)$
NOTE 1 See EN 1993-1-1:2022, 8.3.3(9) for n_z. NOTE 2 See EN 1993-1-1:2022, 8.3.3(10) and EN 1993-1-1:2022, Table 8.9 for C_{mz} and C_{mLT}.		

9 Serviceability limit states

9.1 General

(1) The requirements for serviceability given in EN 1993-1-1:2022, Clause 9 should be applied for stainless steels.

(2) Deflections in members should be estimated in accordance with 9.2.

(3) If visual distortions of the cross-section are unacceptable, the compressive stress $\sigma_{\mathrm{com,Ed,ser}}$ in the cross-section under serviceability loading should be limited by the elastic local and distortional buckling stress of the full cross-section, which may be determined numerically or, in the case of the local buckling stress, according to [11]. Conservatively, for sections comprising flat plates, the local buckling stress may be determined for the most slender constituent plate element of the cross-section using Formula (B.5). For circular hollow sections, the local buckling stress may be determined using Formula (B.7). For sections with edge or intermediate stiffeners, the distortional buckling stress may be determined in accordance with prEN 1993-1-3:2022, 7.3.2.

9.2 Determination of deflections

(1) The effects of the non-linear stress-strain behaviour of stainless steels, and the effectiveness of the cross-section, should be taken into account in estimating deflections.

NOTE: Guidance for the description of the non-linear material behaviour of annealed material is given in prEN 1993-1-14:2022.

(2) For cross-sections susceptible to local or distortional buckling under serviceability loading, the effective moment of inertia should be determined in accordance with prEN 1993-1-3:2022, 9.1(2) based on the modulus of elasticity E.

(3) In the case of members subject to shear lag, the effective cross-section may be based on effective widths determined using prEN 1993-1-5:2022, 5.2.1.

(4) Deflections should be estimated using the secant modulus of elasticity $E_{\mathrm{s,ser}}$ determined taking account of the stresses in the member under the load combination for the relevant serviceability limit state.

(5) The value of the secant modulus of elasticity $E_{\mathrm{s,ser}}$ may be calculated from Formula (9.1):

$$E_{\mathrm{s,ser}} = \frac{(E_{\mathrm{s,1}} + E_{\mathrm{s,2}})}{2} \tag{9.1}$$

where

$E_{\mathrm{s,1}}$ is the secant modulus corresponding to the stress σ_1 in the tension flange;

$E_{\mathrm{s,2}}$ is the secant modulus corresponding to the stress σ_2 in the compression flange.

(6) The values of $E_{\mathrm{s,1}}$ and $E_{\mathrm{s,2}}$ for the appropriate serviceability design stress $\sigma_{i,\mathrm{Ed,ser}}$ may be estimated using Formula (9.2):

$$E_{\mathrm{s},i} = \frac{E}{1+0{,}002\frac{E}{\sigma_{i,\mathrm{Ed,ser}}}\left(\frac{\sigma_{i,\mathrm{Ed,ser}}}{f_{\mathrm{y}}}\right)^{n}} \tag{9.2}$$

with

i = 1 or 2.

(7) The value of the coefficient n may be taken from Table 9.1.

(8) As a simplification, the variation of $E_{s,ser}$ along the length of the member may be neglected and the minimum value of $E_{s,ser}$ for that member (corresponding to the maximum values of the stresses $\sigma_{1,Ed,ser}$ and $\sigma_{2,Ed,ser}$ in the member) may be used throughout its length.

NOTE The use of the secant modulus in (5) or (8) tends to give conservative estimations of the deflection when the maximum stress in the member is $\sigma_{i,Ed,ser} > 0{,}65 f_y$. More accurate analytical expressions for estimating the deflection under typical loading conditions are given in [13].

(9) Alternatively, deflections may be estimated using the finite element methods given in prEN 1993-1-14:2022.

Table 9.1 — Values of *n*

Steel grade	**Coefficient *n***
Ferritic	14
Austenitic	7
Duplex	8

10 Connection design

10.1 General

(1) The provisions given in EN 1993-1-8 should be applied for stainless steels, except where modified or superseded by the provisions given in 10.2 and 10.3.

NOTE: Information on durability is given in Annex A.

(2) The design of connections for stainless steel sheets using self-tapping screws should be in accordance with EN 1993-1-3 except that the pull-out strength should be determined by testing.

(3) The ability of a self-tapping screw to drill and form threads in stainless steel should be demonstrated by tests unless sufficient experience is available.

NOTE Testing of fasteners is described in [14]. Additional information on testing procedures for fastenings is given in [15].

10.2 Bolted connections

(1) The bearing resistance of a bolted connection should be determined either on the basis of a strength or a deformation criterion. The strength criterion may be used provided deformation at the bolt hole under serviceability loading is not a design consideration.

(2) The bearing resistance for an individual fastener should be calculated from Formula (10.1):

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} \tag{10.1}$$

where

α_b is the bearing coefficient in the direction of load transfer given in Table 10.1 or Table 10.2;

k_1 is the bearing coefficient in the direction perpendicular to load transfer given in Table 10.1 or Table 10.2;

d is the bolt diameter;

d_0 is the hole diameter for the bolt;

e_1 is the end distance from the centre of a bolt hole to the adjacent end of any part, measured in the direction of load transfer;

e_2 is the edge distance from the centre of a bolt hole to the adjacent edge of any part, measured at right angles to the direction of load transfer;

p_1 is the spacing between centres of bolts in a line in the direction of load transfer;

p_2 is the spacing measured perpendicular to the load transfer direction between adjacent lines of bolts;

t is the thickness of the connected plate;

f_u is the ultimate tensile strength of the connected plates.

Table 10.1 — α_b and k_1 coefficients ($t > 4$ mm)

	Bearing coefficient α_b	**Bearing coefficient k_1**
Strength criterion	For end bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5e_1}{6d_0}\right\}$ For inner bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5p_1}{12d_0}\right\}$	For edge bolts: $k_1 = \begin{cases} 1{,}0 & \text{if } \min\left\{\frac{e_2}{d_0},\ \frac{p_2}{2d_0}\right\} > 1{,}5 \\ 0{,}8 & \text{if } \min\left\{\frac{e_2}{d_0},\ \frac{p_2}{2d_0}\right\} \leq 1{,}5 \end{cases}$ For inner bolts: $k_1 = \begin{cases} 1{,}0 & \text{if } \left(\frac{p_2}{2d_0}\right) > 1{,}5 \\ 0{,}8 & \text{if } \left(\frac{p_2}{2d_0}\right) \leq 1{,}5 \end{cases}$
Deformation criterion	For end bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5e_1}{4d_0}\right\}$ For inner bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5p_1}{8d_0}\right\}$	For edge and inner bolts: $k_1 = 0{,}5$

Table 10.2 — α_b and k_1 coefficients ($t \leq 4$ mm)

	Bearing coefficient α_b	Bearing coefficient k_1
Strength criterion	**Inner sheet in a double shear connection**	
	For end bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5e_1}{6d_0}\right\}$ For inner bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5p_1}{12d_0}\right\}$	For edge bolts: $k_1 = \begin{cases} 1{,}0 & \text{if} \min\left\{\frac{e_2}{d_0},\ \frac{p_2}{2d_0}\right\} > 1{,}5 \\ 0{,}8 & \text{if} \min\left\{\frac{e_2}{d_0},\ \frac{p_2}{2d_0}\right\} \leq 1{,}5 \end{cases}$ For inner bolts: $k_1 = \begin{cases} 1{,}0 & \text{if} \left(\frac{p_2}{2d_0}\right) > 1{,}5 \\ 0{,}8 & \text{if} \left(\frac{p_2}{2d_0}\right) \leq 1{,}5 \end{cases}$
	Single shear connections and outer sheet in double shear connection	
	For end bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5e_1}{4d_0}\right\}$ For inner bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5p_1}{8d_0}\right\}$	For edge and inner bolts: $k_1 = 0{,}64$
Deformation criterion	**Single and double shear connections**	
	For end bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5e_1}{4d_0}\right\}$ For inner bolts: $\alpha_b = \min\left\{2{,}5,\ \frac{5p_1}{8d_0}\right\}$	For edge and inner bolts: $k_1 = 0{,}5$

(3) The shear resistance of a bolt, $F_{v,Rd}$ should be calculated from Formula (10.2):

$$F_{v,Rd} = \frac{\alpha f_{ub} A}{\gamma_{M2}} \tag{10.2}$$

where

- A is the gross cross-section area of the bolt (if the shear plane passes through the unthreaded portion of the bolt), or the tensile stress area of the bolt (if the shear plane passes through the threaded portion of the bolt);
- f_{ub} is the ultimate tensile strength of the bolt, see Table 5.5.
- α is given in Table 10.3 and applies whether the shear plane passes through the unthreaded or the threaded portions of the bolt.

Table 10.3 —Values of α

Property Class	Austenitic	Duplex
50	0,8	-
70	0,7	0,8
80	0,7	0,7
100	0,6	0,6

(4) The tension resistance of a bolt, $F_{\mathrm{t,Rd}}$ should be calculated from Formula (10.3):

$$F_{t,\mathrm{Rd}} = \frac{k_2 f_{\mathrm{ub}} A_s}{\gamma_{\mathrm{M2}}} \tag{10.3}$$

where

k_2 = 0.63 for countersunk bolt, otherwise k_2 = 1.0;

A_s is the tensile stress area of the bolt.

(5) Bolts which are subjected to combined shear and tension should satisfy the criterion (10.4) or (10.5):

If the threads are not excluded from the shear plane:

$$\left(\frac{F_{\mathrm{v,Ed}}}{F_{\mathrm{v,Rd}}}\right)^{1.7} + \left(\frac{F_{\mathrm{t,Ed}}}{F_{\mathrm{t,Rd}}}\right)^{1.7} \leq 1{,}0 \quad \text{but} \quad \frac{F_{\mathrm{t,Ed}}}{F_{\mathrm{t,Rd}}} \leq 1{,}0 \tag{10.4}$$

If the threads are excluded from the shear plane:

$$\left(\frac{F_{\mathrm{v,Ed}}}{F_{\mathrm{v,Rd}}}\right)^{1.7} + \left(\frac{F_{\mathrm{t,Ed}}}{1{,}25 F_{\mathrm{t,Rd}}}\right)^{1.7} \leq 1{,}0 \quad \text{but} \quad \frac{F_{\mathrm{t,Ed}}}{F_{\mathrm{t,Rd}}} \leq 1{,}0 \tag{10.5}$$

(6) The design slip resistance of preloaded bolted connections should be taken as:

$$F_{\mathrm{s,Rd}} = \frac{k_{\mathrm{s}}\, n\, \mu}{\gamma_{\mathrm{M3}}} F_{\mathrm{p,S}} \tag{10.6}$$

$$F_{\mathrm{s,Rd,ser}} = \frac{k_{\mathrm{s}}\, n\, \mu}{\gamma_{\mathrm{M3,ser}}} F_{\mathrm{p,S}} \tag{10.7}$$

where

k_{s} is given in prEN 1993-1-8:2022, Table 5.10;

n is the number of friction planes;

μ is the slip factor obtained either by specific tests for the friction surface based on test specimens representative of the surfaces used in the structure in accordance with EN 1090-2, or when relevant as given in Table 10.4;

$F_{\mathrm{p,S}}$ is the nominal preload level of the bolting assembly given by Formula (10.8).

$$F_{p,S} = 0{,}7 f_{yb} A_s \tag{10.8}$$

where

f_{yb} is the nominal yield strength of the bolt

A_s is the tensile stress area of the bolt

(7) Proof testing according to 5.2.2 is required to ensure that the preload level is achieved by the chosen tightening procedure and parameters.

Table 10.4 — Slip factors μ for friction surfaces

Surface condition[a]		Class	Slip factor μ
Surface finish	**Rz[b]** [μm]		
Surfaces blasted with clean stainless steel grit media	≥ 55	SSA	0,50
	≥ 45	SSB	0,40
Surfaces blasted with clean stainless steel shot media	≥ 35	SSC	0,20
As rolled surfaces	≥ 25	SSD	0,15

NOTE 1 The potential loss of preloading force from its initial value is considered in these slip factor values.

NOTE 2 Care is needed during grit and shot blasting processes to ensure there is no detrimental effect on the corrosion resistance.

[a] The classification of any other surface treatment should be based on test specimens representative of the surfaces used in the structure, following the procedure set out in EN 1090-2:2018, Annex G.

[b] Rz is the surface roughness according to EN ISO 21920-2.

10.3 Design of welds

(1) The design resistance of a fillet weld should be determined using either the Directional Method or the Simplified Method.

(2) For the Directional Method, the design resistance of a fillet weld should be taken as sufficient if the following are both satisfied:

$$\sqrt{\sigma_{\perp}^2 + 3\left(\tau_{\perp}^2 + \tau_{\parallel}^2\right)} \leq \frac{f_u}{\beta_w \gamma_{M2}} \quad \text{and} \quad \sigma_{\perp} \leq \frac{0{,}9 f_u}{\gamma_{M2}} \tag{10.9}$$

where

f_u is the nominal ultimate tensile strength of the part joined, which is of lower strength grade;

β_w is the appropriate correlation factor, taken as follows:

= 0,9 for welds joining austenitic stainless steels

= 1,0 for welds joining duplex or ferritic stainless steels

= 1,0 for welds joining austenitic to duplex or ferritic stainless steels, duplex to ferritic stainless steels, or stainless steels to carbon steel

(3) For the Simplified Method, the design shear strength of a weld should be determined from:

$$f_{\mathrm{vw,d}} = \frac{f_{\mathrm{u}}}{\sqrt{3}\beta_{\mathrm{w}}\gamma_{\mathrm{M2}}} \qquad (10.10)$$

(4) For welding of material in the cold worked condition, the resistance of the parent metal in the heat affected zones of butt welds should be taken as the ultimate tensile strength of the annealed parent metal, but see 10.3(6) also.

(5) For welding of material in the cold worked condition, the filler metal may have lower strength than the parent metal, in which case the design resistance of fillet and butt welds should be based on the nominal ultimate tensile strength of the filler metal and β_{w} should be taken as 1,0. See 10.3(6) also.

(6) In welded joints of cold worked material, annealing of the heat-affected zones may be incomplete, and the actual strength of joints may be higher than that calculated in accordance with 10.3(4) and (5). Under these circumstances, it may be possible to establish higher design properties by tests.

11 Fatigue

(1) For austenitic and duplex stainless steel structures exposed to fatigue actions, 10 of EN 1993-1-1 shall be applied. The rules in 10 of EN 1993-1-1 shall not apply to ferritic stainless steel structures exposed to fatigue actions.

12 Fire resistance

(1) For stainless steel structures exposed to fire, the rules in Annex C of EN 1993-1-2 shall be applied.

Annex A
(normative)

Selection of materials and durability

A.1 Use of this Annex

(1) This Normative Annex contains provisions on the selection of materials and durability.

A.2 Scope and field of Application

(1) This Normative Annex applies to the selection of stainless steels for structural application and assumes the components in question are load bearing.

(2) The procedure does not take account of:

— grade/product availability;

— surface finish requirements, for example for architectural or hygiene reasons;

— methods of joining/connecting.

The type of surface finish may have an important effect on durability. If visual quality is of importance for a given component, an appropriate finish may be specified in accordance with the product standard.

(3) The procedure assumes that the following criteria will be met:

— the service environment will be in the near neutral pH range (pH 4 to 10);

— the structural parts are not directly exposed to, or part of, a chemical process flow stream;

— the service environment is not permanently or frequently immersed in seawater.

If these conditions are not met, specialist advice should be sought.

NOTE For guidance on material selection for fixings into concrete, timber and masonry, see EN 1992, EN 1995 and EN 1996 respectively.

(4) The procedure is suitable for environments found within Europe. The procedure should not be used for regions outside Europe and may be particularly misleading in certain parts of the world such as the Middle East, Far East and Central America.

A.3 Corrosion protection of construction products — Requirements

(1) Provided that the material is selected in accordance with the procedure given in Tables A.1, A.2 and A.3, subject to the limitations in A.4, and there are no additional requirements given in A.5 to A.7, stainless steel members and fasteners require no applied corrosion protection treatment to ensure satisfactory durability.

A.4 Selection of materials

(1)The procedure involves the following steps:

— determination of the Corrosion Resistance Factor (CRF) for the environment (Table A.1);

— determination of the Corrosion Resistance Class (CRC) from the CRF (Table A.2).

Table A.3 gives grades which have a suitable corrosion resistance for the service environment. The choice of specific grade will depend on other factors in addition to corrosion resistance, such as strength and availability in the required product form. Specification of the material by CRC and Strength Class (from Table 5.1) e.g. CRC III and SC450, or CRC and yield strength, e.g. CRC III and f_y = 450 N/mm^2, is sufficient to allow the supplier to recommend the actual grade.

(2) The procedure applies to components exposed in external environments. For components in internally controlled environments, the CRF is 1,0.

An internally controlled environment is an environment which is either air-conditioned, heated or contained within closed doors. Multi-storey car parks, loading bays or other structures with large openings should be considered as external environments.

NOTE Indoor swimming pools are special cases of internal environments covered by A.5.

(3) The CRF depends on the severity of the environment and is calculated as follows:

$$\text{CRF} = F_1 + F_2 + F_3$$

where

F_1 = Risk of exposure to chlorides from salt water or deicing salts;

F_2 = Risk of exposure to sulfur dioxide;

F_3 = Cleaning regime or exposure to washing by rain.

(4) The value of F_1 for applications on the coastline depends on the particular location in Europe and is derived from experience with existing structures, corrosion test data and chloride distribution data. The large range of environments within Europe means that in some cases the calculated CRF will be conservative. The conservatism is due to the assumption of exposure to consistently high deposition rates of airborne chlorides from the sea. Where justified by a detailed site specific assessment undertaken by an appropriate specialist, the CRC may be reduced by one class. The assessment should take account of factors which may reduce chloride deposition including, but not limited to, proximity to the open sea, sheltering by ground topography or buildings, prevailing wind direction and breaking wave activity.

NOTE The National Annex can specify whether a less severe CRF may be chosen when validated local operating experience or test data support such a choice.

(5) Different parts of the same structure may have different exposure conditions, for example one part may be fully exposed and another part fully sheltered. Each exposure case should be assessed separately.

(6) The procedure assumes that the requirements of EN 1090-2 are followed in relation to:

— welding procedures and post weld cleaning,

— avoidance or removal and cleaning of contamination of the stainless steel surfaces after thermal or mechanical cutting.

Failure to do so may reduce the corrosion resistance of welded parts.

Table A.1 — Determination of Corrosion Resistance Factor CRF = $F_1 + F_2 + F_3$

Risk of exposure to chlorides from salt water or deicing salts, F_1		
1	Internally controlled environment	
0	Low risk of exposure	M > 10 km or S > 0,1 km
-3	Medium risk of exposure	1 km < M ≤ 10 km or 0,01 km < S ≤ 0,1 km
-7	High risk of exposure	0,25 km < M ≤ 1 km or S ≤ 0,01 km
-10	Very high risk of exposure	Road tunnels where deicing salt is used or where vehicles might carry deicing salts into the tunnel
-10	Very high risk of exposure	M ≤ 0,25 km[a] North Sea coast of Germany and all Baltic coastal areas
-15	Very high risk of exposure	M ≤ 0,25 km[a] Atlantic coast line of Portugal, Spain and France. English Channel and North Sea Coastline of UK, France, Belgium, Netherlands and Southern Sweden. All other coastal areas of UK, Norway, Denmark and Ireland. Mediterranean Coast
NOTE 1 M is distance from the sea and S is distance from roads with deicing salts. [a] The distance M<0,25 km assumes the structure is not sheltered by ground topography. If the topography provides partial shelter to the structure, experience shows a grade from one lower class may be used. Examples of sheltering include structures built over inlets or estuaries with limited wave heights, physical barriers including trees, hills and other buildings within the 0,25 km zone.		
Risk of exposure to sulfur dioxide, F_2		
0	Low risk of exposure	< 10 μg/m³ average gas concentration
-5	Medium risk of exposure	10 - 90 μg/m³ average gas concentration
-10	High risk of exposure	90 - 250 μg/m³ average gas concentration
NOTE 2 For European coastal environments the sulfur dioxide concentration is usually low. For inland environments the sulfur dioxide concentration is either low or medium. The high classification is unusual and associated with particularly heavy industrial locations or specific environments such as road tunnels. Sulfur dioxide concentration may be evaluated according to the method in EN ISO 9225. Ferritic stainless steels demonstrate increased sensitivity to sulfur dioxide.		
Cleaning regime or exposure to washing by rain, F_3		
0	Fully exposed to washing by rain	
-2	Specified cleaning regime	
-7	No washing by rain or No specified cleaning	
NOTE 1 If $F_1 + F_2 \geq 0$, then $F_3 = 0$ NOTE 2 If the component is to be regularly inspected for any signs of corrosion and cleaned, this should be made clear to the user in written form. The inspection, cleaning method and frequency should be specified. The more frequently cleaning is carried out, the greater the benefit. The frequency should not be less than every 3 months. Where cleaning is specified it should apply to all parts of the structure, and not just those easily accessible and visible.		

Table A.2 — Determination of Corrosion Resistance Class CRC

Corrosion Resistance Factor (CRF)	Corrosion Resistance Class (CRC)
CRF = 1	I
0 ≥ CRF > −7	II
-7 ≥ CRF > −15	III
-15 ≥ CRF ≥ −20	IV
CRF < −20	V

Table A.3 — Grades in each Corrosion Resistance Class CRC

Corrosion resistance class CRC[a,b,c]				
I	**II**	**III**	**IV**	**V**
1.4003	1.4301	1.4401	1.4439	1.4565
1.4016	1.4307	1.4404	1.4462	1.4529
1.4512	1.4311	1.4435	1.4539	1.4547
	1.4541	1.4571	1.4662	1.4410
	1.4318	1.4429		1.4501
	1.4306	1.4432		1.4507
	1.4567	1.4162		
	1.4482	1.4362		
	1.4621	1.4062		
	1.4622	1.4578		
	1.4509			
	1.4521			
	1.4420			

[a] A grade from a higher class may be used in place of the class indicated by the CRF.

[b] The corrosion resistant classes are only intended for use with this grade selection procedure and are only applicable to structural applications.

[c] The fracture toughness of ferritic stainless steels should be checked for external applications, see 5.1.3.

A.5 Swimming pool environments

(1) Only the steel grades given in Table A.4 shall be used for load bearing parts exposed to atmospheres above indoor swimming pools.

Table A.4— Steel grades for indoor swimming pool atmospheres

Load bearing parts in swimming pool atmospheres	Corrosion resistance class CRC
Load-bearing members which are regularly cleaned[a]	CRC III or CRC IV
Load-bearing members which are not regularly cleaned	CRC V (excluding 1.4410, 1.4501 and 1.4507)
All fixings, fasteners and threaded parts	CRC V (excluding 1.4410, 1.4501 and 1.4507)
NOTE The National Annex can specify if less frequent cleaning is permitted.	
[a] If the component is to be regularly inspected for any signs of corrosion and cleaned, this should be made clear to the user in written form. The inspection, cleaning method and frequency should be specified. The more frequently cleaning is carried out, the greater the benefit. The frequency should not be less than every week. Where cleaning is specified, it should apply to all parts of the structure, and not just those easily accessible and visible.	

A.6 Corrosion protection of connections with other metals

(1) Bimetallic corrosion may occur if dissimilar metals are in electrical contact and the contact area is exposed to an electrolyte (e.g. water or soil). Bimetallic corrosion may result in additional corrosion of one of the metals unless it is protected or electrically isolated from the other metal.

(2) If necessary, bimetallic corrosion should be prevented by isolating the stainless steel electrically from the other metal. Electrical isolation may be achieved by the use of insulating washers and bushes on both sides of the joint or by protective coatings applied to the non-stainless steel parts.

(3) Special measures should be taken to ensure the durability of welds between stainless steel and other metals (usually carbon steel), for example the weld should be painted and the paint continued at least 75 mm onto the stainless steel.

A.7 Galvanizing and contact with molten zinc

(1) Hot-dip galvanizing of components made of stainless steel is not allowed because contact with molten zinc can cause embrittlement of the stainless steel.

(2) Precautions should be taken to ensure that in the event of a fire, molten zinc from galvanised steel cannot drip or run onto the stainless steel and cause embrittlement. Additionally, there is a risk of embrittlement if a stainless steel component is joined to a carbon steel component which subsequently undergoes hot-dip galvanizing.

Annex B
(normative)

Continuous strength method

B.1 Use of this Annex

(1) This Normative Annex contains additional provisions for determining the resistance of cross-sections which take advantage of the beneficial effect of partial plastification and strain hardening.

B.2 Scope and field of Application

(1) This Normative Annex applies to I-sections, channels, T-sections, angles and rectangular and circular hollow sections that satisfy the cross-section slenderness limits specified in B.5, and the member slenderness limits specified in 8.3.2.1(3) and 8.3.3.1(3).

B.3 General

(1)For cold-formed cross-sections, the average enhanced yield strength f_{ya} and the average ultimate tensile strength f_{ua} of the cross-section calculated according to 5.1.2.3 may be used in place of f_y and f_u, respectively, in this Annex.

B.4 Material modelling

(1) The CSM bi-linear elastic, linear hardening material model is shown in Figure B.1 while the material coefficients (C_1, C_2 and C_3) which are used to define the material model are given in Table B.1.

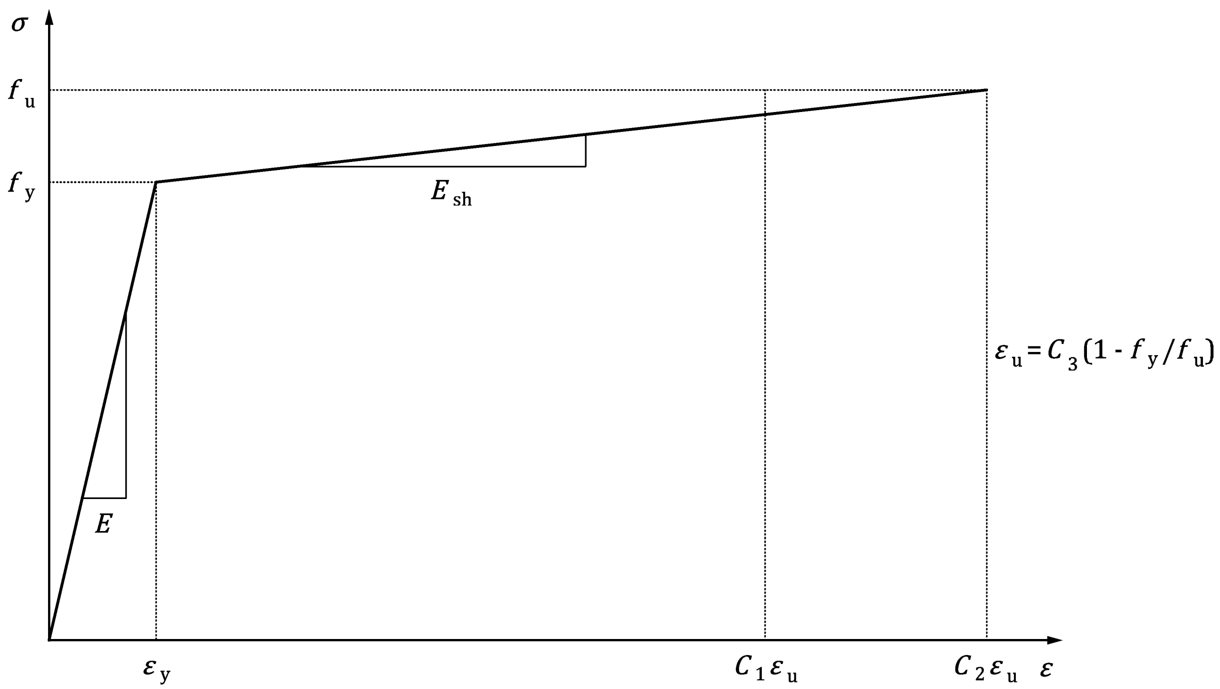


Figure B.1 — CSM bi-linear elastic, linear hardening material model

The terms in Figure B.1 are defined as:

f_y is the yield strength

ε_y is the elastic strain at the yield strength, taken as f_y/E

E is the modulus of elasticity

E_{sh} is the strain hardening modulus, taken as:

$$E_{sh} = \frac{f_u - f_y}{C_2 \varepsilon_u - \varepsilon_y} \tag{B.1}$$

f_u is the ultimate tensile strength

ε_u is the ultimate strain, corresponding to the ultimate tensile strength f_u, taken as:

$$\varepsilon_u = C_3 \left(1 - f_y/f_u\right) \tag{B.2}$$

Table B.1 — CSM material model coefficients

Stainless steel	C_1	C_2	C_3
Austenitic	0,10	0,16	1,00
Duplex	0,10	0,16	1,00
Ferritic	0,40	0,45	0,60

B.5 Cross-section deformation capacity

B.5.1 Base curve

(1) The normalized deformation capacity of sections comprising flat plates and circular hollow sections should be determined following Formulae (7.9) and (B.3), respectively, based on the slenderness of the cross-section calculated in accordance with B.5.2. The upper slenderness limit of $\bar{\lambda}_{p,cs} \leq 1,0$ in Formula (7.9) may be relaxed to $\bar{\lambda}_{p,cs} \leq 1,6$ when used with the resistance formulae given in this Annex.

For circular hollow sections

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \begin{cases} \dfrac{4,44 \times 10^{-3}}{\bar{\lambda}_{c,cs}^{\;4,5}} \leq \min\left(\Omega, \quad \dfrac{C_1 \varepsilon_u}{\varepsilon_y}\right) & \text{for } \bar{\lambda}_{c,cs} \leq 0,30 \\ \left(1 - \dfrac{0,224}{\bar{\lambda}_{c,cs}^{\;0,342}}\right) \dfrac{1}{\bar{\lambda}_{c,cs}^{\;0,342}} & \text{for } 0,30 < \bar{\lambda}_{c,cs} \leq 0,60 \end{cases} \tag{B.3}$$

where

ε_{csm} is the CSM strain limit;

$\bar{\lambda}_{p,cs}$ is the cross-section slenderness for sections comprising flat plates(e.g. I-sections);

$\bar{\lambda}_{c,cs}$ is the cross-section slenderness for circular hollow sections;

Ω is defined in 7.4.3.5.

B.5.2 Cross-section slenderness

(1) For a section comprising flat plates, the cross-section slenderness $\bar{\lambda}_{p,cs}$ is defined as:

$$\bar{\lambda}_{p,cs} = \sqrt{f_y / \sigma_{cr,cs}} \tag{B.4}$$

In which:

$\sigma_{cr,cs}$ is the elastic local buckling stress of the full cross-section. This can be determined from numerical methods or the analytical expressions given in [11].

(2) Conservatively, $\sigma_{cr,cs}$ may be taken as the elastic local buckling stress of the most slender constituent plate element of the cross-section $\sigma_{cr,p}$, determined using Formula (B.5):

$$\sigma_{cr,p} = \frac{k_\sigma \pi^2 E t^2}{12(1-\upsilon^2)\bar{b}^2} \tag{B.5}$$

where

$\bar{b}$ is the plate element flat width (see 8.2.2(5))

t is the plate element thickness

υ is Poisson's ratio

k_σ is the buckling factor corresponding to the stress ratio ψ and boundary conditions as given in prEN 1993-1-5:2022, 6.4.1 for I nternal and outstand elements.

(3) For a circular hollow section, the cross-section slenderness $\bar{\lambda}_{c,cs}$ is defined as:

$$\bar{\lambda}_{c,cs} = \sqrt{f_y / \sigma_{cr,c}} \tag{B.6}$$

In which:

$\sigma_{cr,c}$ is the elastic buckling stress of the full cross-section of the circular hollow section. This can be determined numerically or for members in compression, bending or combinations thereof, it may be determined using Formula (B.7):

$$\sigma_{cr,c} = \frac{E}{\sqrt{3(1-\upsilon^2)}} \frac{2t}{d} \tag{B.7}$$

where

d is the cross-section diameter

t is the cross-section thickness

B.6 Resistance of cross-sections

B.6.1 Tension

(1) For sections comprising flat plates and circular hollow sections, the design tension resistance of the cross-section without holes should be determined as:

$$N_{t,Rd} = N_{csm,t,Rd} = \frac{A f_{csm,t}}{\gamma_{M0}} \tag{B.8}$$

where

A is the cross-sectional area

$f_{\mathrm{csm,t}}$ is the design stress corresponding to $\varepsilon_{\mathrm{csm,t}}$, given by:

$$f_{\mathrm{csm,t}} = f_{\mathrm{y}} + E_{\mathrm{sh}}\varepsilon_{\mathrm{y}}(\varepsilon_{\mathrm{csm,t}}/\varepsilon_{\mathrm{y}} - 1) \quad \text{(B.9)}$$

$$\varepsilon_{\mathrm{csm,t}}/\varepsilon_{\mathrm{y}} = \min\left\{15\ ;\ \frac{C_1\varepsilon_{\mathrm{u}}}{\varepsilon_{\mathrm{y}}}\right\} \quad \text{(B.10)}$$

(2) For sections comprising flat plates and circular hollow sections with holes, the design net-section resistance should be determined according to prEN 1993-1-3:2022, 8.1.2 or EN 1993-1-1:2022, 8.2.3 as appropriate.

B.6.2 Compression

(1) For sections comprising flat plates and circular hollow sections, the design resistance of the cross-section for uniform compression should be determined as follows:

$$N_{\mathrm{c,Rd}} = N_{\mathrm{csm,Rd}} = \frac{\varepsilon_{\mathrm{csm}}}{\varepsilon_{\mathrm{y}}}\frac{Af_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} \qquad \text{for } \varepsilon_{\mathrm{csm}}/\varepsilon_{\mathrm{y}} < 1{,}0 \quad \text{(B.10)}$$

$$N_{\mathrm{c,Rd}} = N_{\mathrm{csm,Rd}} = \frac{Af_{\mathrm{csm}}}{\gamma_{\mathrm{M0}}} \qquad \text{for } \varepsilon_{\mathrm{csm}}/\varepsilon_{\mathrm{y}} \geq 1{,}0 \quad \text{(B.11)}$$

where

A is the cross-sectional area

f_{csm} is the design stress corresponding to $\varepsilon_{\mathrm{csm}}$, given by:

$$f_{\mathrm{csm}} = f_{\mathrm{y}} + E_{\mathrm{sh}}\varepsilon_{\mathrm{y}}(\varepsilon_{\mathrm{csm}}/\varepsilon_{\mathrm{y}} - 1) \quad \text{(B.12)}$$

B.6.3 Bending

(1) The bending moment resistance $M_{\mathrm{csm,Rd}}$ given in B.6.3.1 and B.6.3.2 may be used for member slenderness $\bar{\lambda}_{\mathrm{LT}} \leq 0{,}2$. For the case in which $M_{\mathrm{csm,Rd}}$ is greater than the bending moment resistance determined according to 8.2.4, a linear reduction from $M_{\mathrm{csm,Rd}}$ to $M_{\mathrm{c,Rd}}$ (determined from 8.2.4) between $\bar{\lambda}_{\mathrm{LT}} \leq 0{,}2$ and $\bar{\lambda}_{\mathrm{LT}} \leq 0{,}4$ may be used.

B.6.3.1 Bending about an axis of symmetry

(1) For doubly symmetric sections (e.g. I-sections, rectangular and circular hollow sections) and mono-symmetric sections (e.g. channel sections and T-sections) in bending about an axis of symmetry, the cross-section bending resistance should be determined using Formulae (B.13) or (B.14) based on the CSM strain limit $\varepsilon_{\mathrm{csm}}$ obtained in accordance with B.5.1.

$$M_{\mathrm{c,Rd}} = M_{\mathrm{csm,Rd}} = \frac{\varepsilon_{\mathrm{csm}}}{\varepsilon_{\mathrm{y}}}\frac{W_{\mathrm{el}}f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} \qquad \text{for } \varepsilon_{\mathrm{csm}}/\varepsilon_{\mathrm{y}} < 1{,}0 \quad \text{(B.13)}$$

$$M_{\mathrm{c,Rd}} = M_{\mathrm{csm,Rd}} = \frac{W_{\mathrm{pl}}f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}\left[1 + \frac{E_{\mathrm{sh}}}{E}\frac{W_{\mathrm{el}}}{W_{\mathrm{pl}}}\left(\frac{\varepsilon_{\mathrm{csm}}}{\varepsilon_{\mathrm{y}}} - 1\right) - \left(1 - \frac{W_{\mathrm{el}}}{W_{\mathrm{pl}}}\right)\Big/\left(\frac{\varepsilon_{\mathrm{csm}}}{\varepsilon_{\mathrm{y}}}\right)^{\alpha}\right] \qquad \text{for } \varepsilon_{\mathrm{csm}}/\varepsilon_{\mathrm{y}} \geq 1{,}0 \quad \text{(B.14)}$$

where

W_{el} is the cross-section elastic section modulus;

W_{pl} is the cross-section plastic section modulus;

α is the CSM bending parameter, as given in Table B.2.

B.6.3.2 Bending about an axis that is not one of symmetry

(1) For asymmetric sections (e.g. angles) and mono-symmetric sections (e.g. channel sections and T-sections) in bending about an axis that is not one of symmetry, the CSM compressive strain limit ε_{csm} should be determined in accordance with Formula (7.9). The corresponding outer-fibre tensile strain $\varepsilon_{csm,t}$ may then be determined assuming a linearly-varying through-depth strain distribution and the design bending moment resistance calculated as follows:

- Initially, $\varepsilon_{csm,t}$ may be calculated based on the location of the elastic neutral axis (ENA). The maximum design strain $\varepsilon_{csm,max}$ should then be taken as the maximum of ε_{csm} and $\varepsilon_{csm,t}$.

- If $\varepsilon_{csm,max} \leq \varepsilon_y$, the use of the ENA in the calculation of $\varepsilon_{csm,t}$ may be considered to be appropriate and the design bending moment resistance should be calculated using Formula (B.13) with ε_{csm} replaced by $\varepsilon_{csm,max}$.

- If $\varepsilon_{csm,max} > \varepsilon_y$, the location of the design neutral axis should be recalculated based on cross-section equilibrium or, as an approximation, it may be considered to lie at mid-distance between the elastic and plastic neutral axes. $\varepsilon_{csm,t}$ and $\varepsilon_{csm,max}$ should then be recalculated using the new location of the neutral axis.

- The bending moment resistance should then be determined using Formula (B.14) with ε_{csm} replaced by $\varepsilon_{csm,max}$ and using the values of the bending parameter α taken from Table B.2.

Table B.2 — CSM bending parameter α

Cross-section type	Axis of bending	Aspect ratio	α
Rectangular hollow section	Any	Any	2,0
Circular hollow section	Any	–	2,0
I-section	Major	Any	2,0
	Minor	Any	1,2
Channel section	Major	Any	2,0
	Minor	$h/b < 2$	1,5
		$h/b \geq 2$	1,0
T-section	Major	$h/b < 1$	1,0
		$h/b \geq 1$	1,5
	Minor	Any	1,2
Equal angle	Any	-	1,0
Unequal angle	Major	Any	1,5
	Minor	Any	1,0

B.6.4 Combined bending and axial force

B.6.4.1 Rectangular hollow sections subject to combined loading

(1) For rectangular hollow sections with $\bar{\lambda}_{\mathrm{p,cs}} \leq 0{,}60$, subjected to uniaxial bending in combination with axial compression, the criteria given by Formulae (B.15) and (B.16) should be satisfied for bending about the major axis and minor axis, respectively. For biaxial bending in combination with compression, the criteria given by Formula (B.17) should be satisfied.

$$M_{\mathrm{y,Ed}} \leq M_{\mathrm{N,csm,y,Rd}} = M_{\mathrm{csm,y,Rd}} \frac{(1-n_{\mathrm{csm}})}{(1-0{,}5a_{\mathrm{w}})} \leq M_{\mathrm{csm,y,Rd}} \tag{B.15}$$

$$M_{\mathrm{z,Ed}} \leq M_{\mathrm{N,csm,z,Rd}} = M_{\mathrm{csm,z,Rd}} \frac{(1-n_{\mathrm{csm}})}{(1-0{,}5a_{\mathrm{f}})} \leq M_{\mathrm{csm,z,Rd}} \tag{B.16}$$

$$\left[\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{N,csm,y,Rd}}}\right]^{\alpha_{\mathrm{csm}}} + \left[\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{N,csm,z,Rd}}}\right]^{\beta_{\mathrm{csm}}} \leq 1 \tag{B.17}$$

where

$M_{\mathrm{y,Ed}}$	is the design bending moment about major (y-y) axis
$M_{\mathrm{z,Ed}}$	is the design bending moment about minor (z-z) axis
$M_{\mathrm{N,csm,y,Rd}}$	is the reduced CSM bending moment resistance about major (y-y) axis
$M_{\mathrm{N,csm,z,Rd}}$	is the reduced CSM bending moment resistance about minor (z-z) axis
a_{w}	is the ratio of the web area to the gross cross-section area
a_{f}	is the ratio of the flange area to the gross cross-section area
n_{csm}	is the ratio of the design compression force N_{Ed} to the CSM compression resistance $N_{\mathrm{csm,Rd}}$
α_{csm} and β_{csm}	are the interaction coefficients for biaxial bending, equal to $1{,}66/(1-1{,}13{n_{\mathrm{csm}}}^2)$.

(2)For rectangular hollow sections with $\bar{\lambda}_{\mathrm{p,cs}} > 0{,}60$, the criteria given by Formula (B.18) should be satisfied.

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{csm,Rd}}} + \frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{csm,y,Rd}}} + \frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{csm,z,Rd}}} \leq 1 \tag{B.18}$$

B.6.4.2 I-sections subject to combined loading

(1) For I-sections with $\bar{\lambda}_{\mathrm{p,cs}} \leq 0{,}60$, subjected to uniaxial bending in combination with axial compression, the criteria given by Formulae (B.19), (B.20) and (B.21) should be satisfied for bending about the major axis and minor axis, respectively. For biaxial bending in combination with compression, the criteria given by Formula (B.17) should be satisfied.

$$M_{\mathrm{y,Ed}} \leq M_{\mathrm{N,csm,y,Rd}} = M_{\mathrm{csm,y,Rd}} \frac{(1-n_{\mathrm{csm}})}{(1-0{,}5a)} \leq M_{\mathrm{csm,y,Rd}} \tag{B.19}$$

$$M_{\mathrm{z,Ed}} \leq M_{\mathrm{N,csm,z,Rd}} = M_{\mathrm{csm,z,Rd}} \qquad \text{for } n_{\mathrm{csm}} \leq a \tag{B.20}$$

$$M_{\mathrm{z,Ed}} \leq M_{\mathrm{N,csm,z,Rd}} = M_{\mathrm{csm,z,Rd}} \left[1 - \left(\frac{n_{\mathrm{csm}}-a}{1-a}\right)^2\right] \leq M_{\mathrm{csm,z,Rd}} \quad \text{for} \quad n_{\mathrm{csm}} > a \tag{B.21}$$

where

$a \quad = (A - 2bt_{\mathrm{f}})/A$

$\alpha_{\mathrm{csm}} = 2$ and $\beta_{\mathrm{csm}} = 5n_{\mathrm{csm}}$ but $\beta_{\mathrm{csm}} \geq 1$ are the interaction coefficients for biaxial bending.

(2) For I-sections with $\bar{\lambda}_{\mathrm{p,cs}} > 0{,}60$, the criteria given by Formula (B.18) should be satisfied.

B.6.4.3 Circular hollow sections subject to combined loading

(1) For circular hollow sections with $\bar{\lambda}_{\mathrm{c,cs}} \leq 0{,}27$, the criteria in Formula (B.22) should be satisfied:

$$M_{\mathrm{Ed}} \leq M_{\mathrm{N,csm,Rd}} = M_{\mathrm{csm,Rd}}(1 - n_{\mathrm{csm}}{}^{1{,}7}) \tag{B.22}$$

(2) For circular hollow sections with $\bar{\lambda}_{\mathrm{c,cs}} > 0{,}27$, the criteria in Formula (B.23) should be satisfied:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{csm,Rd}}} + \frac{M_{\mathrm{Ed}}}{M_{\mathrm{csm,Rd}}} \leq 1 \tag{B.23}$$

Bibliography

References contained in recommendations (i.e. "should" clauses)

The following documents are referred to in the text in such a way that some or all of their content constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative documents could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

[1] EN 10088 (all parts), *Stainless steels*

[2] EN ISO 3506-1, *Mechanical properties of corrosion-resistant stainless steel fasteners — Part 1: Bolts, screws and studs with specified grades and property classes*

[3] EN ISO 3506-2, *Mechanical properties of corrosion-resistant stainless steel fasteners — Part 2: Nuts with specified grades and property classes*

[4] EN ISO 7089, *Plain washers — Normal series — Product grade A*

[5] EN ISO 7090, *Plain washers, chamfered — Normal series — Product grade A*

[6] EN 10028-7, *Flat products made of steels for pressure purposes – Part 7: Stainless steels*

[7] EN 10296-2, *Welded circular steel tubes for mechanical and general engineering purposes - Technical delivery conditions – Part 2: Stainless steel*

[8] EN 10297-2, *Seamless circular steel tubes for mechanical and general engineering purposes - Technical delivery conditions – Part 2: Stainless steel*

References contained in permissions (i.e. "may" clauses)

The following documents are referred to in the text in such a way that some or all of their content expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

[9] EN ISO 9225, *Corrosion of metals and alloys — Corrosivity of atmospheres — Measurement of environmental parameters affecting corrosivity of atmospheres*

[10] EN ISO 21920-2, *Geometrical product specifications (GPS) — Surface texture: Profile — Part 2: Terms, definitions and surface texture parameters*

[11] Gardner, L., Fieber, A. and Macorini, L. (2019). Formulae for calculating elastic local buckling stresses of full structural cross-sections. Structures. 17, 2-20.

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